AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

382.

(Vol. XVIII.—May, 1888.)

TEST OF A WROUGHT IRON DOUBLE TRACK FLOOR BEAM.

By ALFRED P. BOLLER, M. Am. Soc. C. E.

READ NOVEMBER 16TH, 1887.

WITH DISCUSSION.

Testing to rupture actual bridge members is always a matter of great scientific interest, and while the record is quite extensive in eye-bars, posts, or small parts, the great cost, time and inconvenience of handling heavy girders, has prevented experiment in that direction. In fact, the writer is unaware of any experiment upon compound riveted beams, on a large scale, as actually used, until the experiment recorded below was made, under his supervision.

The beam was an exact duplicate of those in use on a bridge, about which more or less controversy had arisen, and to settle the different views of "experts" as to their practical safety, the company ordered a test made under, as near as possible, actual conditions of attachment and loading. Plates XXXVI and XXXVII show the form and proportion of the beam, and connection with the posts, together with the position of the track stringers. The actual static loads to which the beam could be subjected by the heaviest engines in use on the road, with weight of

floor is 40 000 pounds at each stringer bearing. The strains computed therefrom being as follows:

Flange strains at m 3 800 pounds per square inch.
" " a 5 700 " "
" " b 6 400 " "
Shear strains in web between a and b, 2 600 pounds per square inch.
" " between a and end.
8 000 pounds per square inch, at least section, or where web is 2 feet four inches deep, or 42 diameters between angle-iron.

Rivets.—All rivets $\frac{7}{8}$ inches diameter or $\frac{15}{16}$ when driven to fill holes—area of section 0.6 square inches, bearing area diameter x $\frac{3}{8}$ plate=0.35 square inches, and for $\frac{1}{2}$-inch plate 0.47 square inches.

Post attachment considering all the 26 rivets doing duty, yields rivet strain as follows,
in shear (single).............. 5 000 pounds per square inch.
and bearing area ($\frac{1}{2}$-inch plate,) 6 600 " "

CONNECTION OF $\frac{3}{8}$ WEB TO FLANGE ANGLES:

Taking the 40 rivets between ends of girder and second stringer, the horizontal strain difference is 162 000 pounds, the rivets being strained 3 400 pounds per square inch double shear, and 11 600 pounds per square inch bearing area.

Taking distance from ends to first stringer, the horizontal strain difference is 105 000 pounds—yielding on 20 rivets, 4 200 pounds per square inch double shear, and 15 000 pounds per square inch bearing area.

Taking a short distance of two feet from ends, the horizontal strain is 70 000 pounds on 10 rivets, giving 5 800 pounds per square inch double shear, and 20 000 pounds per square inch bearing area.

In these girders, the weakness feared was in the end flange riveting and shear in end web, and caused the test recorded below.

The test was recently made at the works of the Keystone Bridge Company, by means of hydraulic power applied at stringer points; convenience made it necessary to make the test with the beam blocked up horizontally on the ground, so that the weight of the beam is necessarily neglected. The beam was connected with a pair of posts, precisely as in the actual structure, between which an additional girder was framed as a reaction base for the rams. Plate XXXVII shows the general arrangements. The hydraulic power was derived from the testing machine plant of the Keystone establishment, and the deflections measured from a fine wire parallel to lower flange, and about 3 inches

therefrom. The diameter of the rams was 10 inches. (Area 78.54 inches.) The record was as follows:

Gauge Reading.	Load on each Ram.	Deflections. b.	Deflections. b'.	Total Load.
565	44 375 pounds.	$\frac{1}{8}$ inch.	$\frac{1}{8}$ inch.	177 500 pounds.
1 130	88 750 "	$\frac{5}{16}$ "	$\frac{5}{16}$ "	355 000 "
1 412	110 900 "	$\frac{3}{8}$ "	$\frac{3}{8}$ "	443 600 "
No permanent set in above.				
1 695	133 125 pounds.	Uncertain.		532 500 "
Permanent set scant $\frac{1}{32}$ of an inch.				
1 980	155 500 "	Not recorded.		622 000 "
Permanent set $\frac{5}{32}$ of an inch.				
2 080—Failure commenced through giving way of angle-irons beginning in a fine seam..............................				653 500 "

—at first bend in lower flange, from end support, the seam being along the root of the angle, which continual pressure tore apart across the angle as shown, when the web commenced to tear like a sheet of paper, in direction and manner as exhibited on Plate XXXVIII herewith (from photograph). From some cause, not apparent, the deflections were not similar at the symmetrical end rams (a), the point where the web failed (left side) being sharply deflected. While the angles showed root fracture at the opposite point, the web did not fail or show indications of so doing, the deflection being on an easy curve. With the extreme yielding of the lower flange angles, the angle brackets connecting girder with posts, commenced to go, tearing likewise along the root, and stripping the heads from the extreme upper rivets as shown. The internal diaphragm connecting the channel sides of the posts, was unaffected.

The rivets connecting the ruptured flange with web appeared as perfect as when driven, and no indication was disclosed, as far as it was possible to tell, of the holes in the web elongating, or any upsetting of bearing surface. There is no telling what the web and rivets would have borne, had not the solid angle-irons given way, at the first bend.

It is to be noted that flange plate, with leg of angle attached thereto, was intact, showing no indication of rupture.

Discussion.—Taking that stage of the experiment when a permanent set was first noted, viz.: $\frac{1}{32}$ of an inch, the recorded load was 532 500 pounds, or as near as may be $3\frac{1}{3}$ times the basis on which the calculations in the first part of this paper were made (40 000 pounds on each stringer or 160 000 pounds total). Applying this ratio to the preceding computations, the iron would be apparently strained as follows:

Flanges	at *m*	3 800 × $3\frac{1}{3}$ = 12 600	pounds per square inch.			
	a	5 700 × " = 19 000	"	"		
	b	6 400 × " = 21 200	"	"		
Web	between *a* and *b*..	2 600 × " = 8 700	"	"		
	least section......	8 000 × " = 26 600	"	"		
Rivets	Post attachment,					
	bearing area......	6 600 × " = 22 000	"	"		
	double shear......	5 000 × " = 16 600	"	"		
	Web and flange connections end rivets,					
	bearing area	20 000 × " = 66 600	"	"		
	double shear.....	5 800 × " = 19 300	"	"		

When failure in angles was first noted, the recorded load was 653 500 pounds, or slightly more than four times the computed basis of load, which would increase the above strains about one-fifth, giving a calculated flange strain when angle failed of some 15 000 pounds per square inch—and bearing area strain on end flange and web rivets about 80 000 pounds per square inch—neither of which could possibly be true, or the web would have torn out from the rivets, and the flanges be perfectly sound, well within elastic limits, although in the last case it is to be noted that the horizontal table of the flange was perfectly sound, the flange failure commencing primarily with a long split along the weld of the angle-iron root, throwing the whole flange duty upon the vertical legs of the angle-irons when a rupture strain was quickly reached. Had the angles been rolled from a solid ingot, or on the German method of developing from a flat, instead of from the ordinary welded pile, the strength of this beam would have been largely increased. The prime weakness in the beam was due therefore to the mode of manufacturing the angle-irons, which were weak along the weld at the root. This was also shown in the end bracket angles uniting the beam to the posts.

The experiment further shows that a plate web is an exceedingly stiff member, much stiffer than is commonly supposed, all formula for proportioning the same being utterly in the fog, the dicta of one expert being about as good as that of another. It demonstrates that the customary method of proportioning rivets, viz., the horizontal component between any two given points, divided by allowable bearing pressure per square inch equals number of rivets required, is not true, and that the friction due to power riveting has enormous value.

This beam was reported to the company interested as practicably safe by the writer, on general considerations, before the experiment was made, and the opinion reaffirmed after the experiment.

PLATE XXXVI
TRANS. AM. SOC. CIV. ENGRS.
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BOLLER ON TEST OF
WROUGHT IRON FLOOR BEAM.

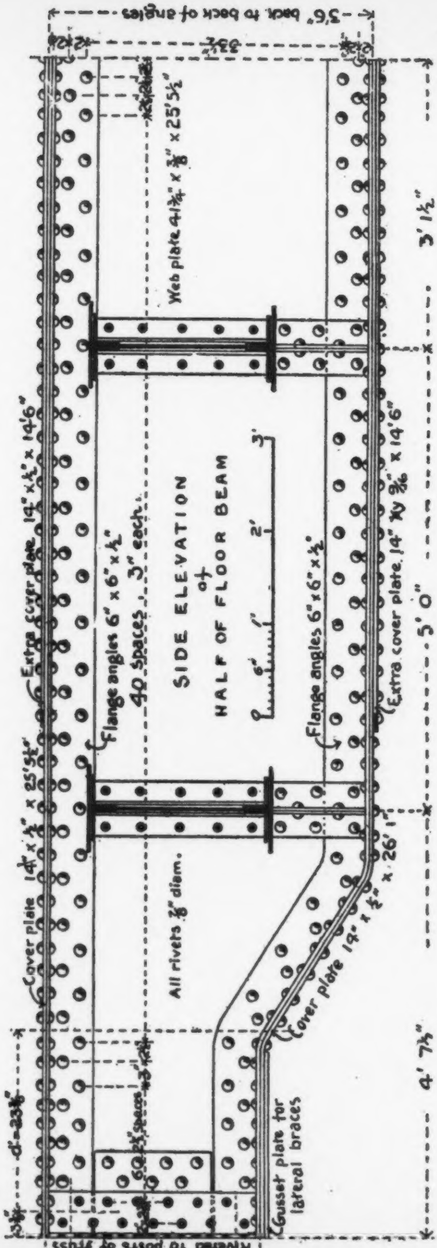


PLATE XXXVII
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XVIII Nº 382.
BOLLER ON TEST OF
WROUGHT IRON FLOOR BEAM.

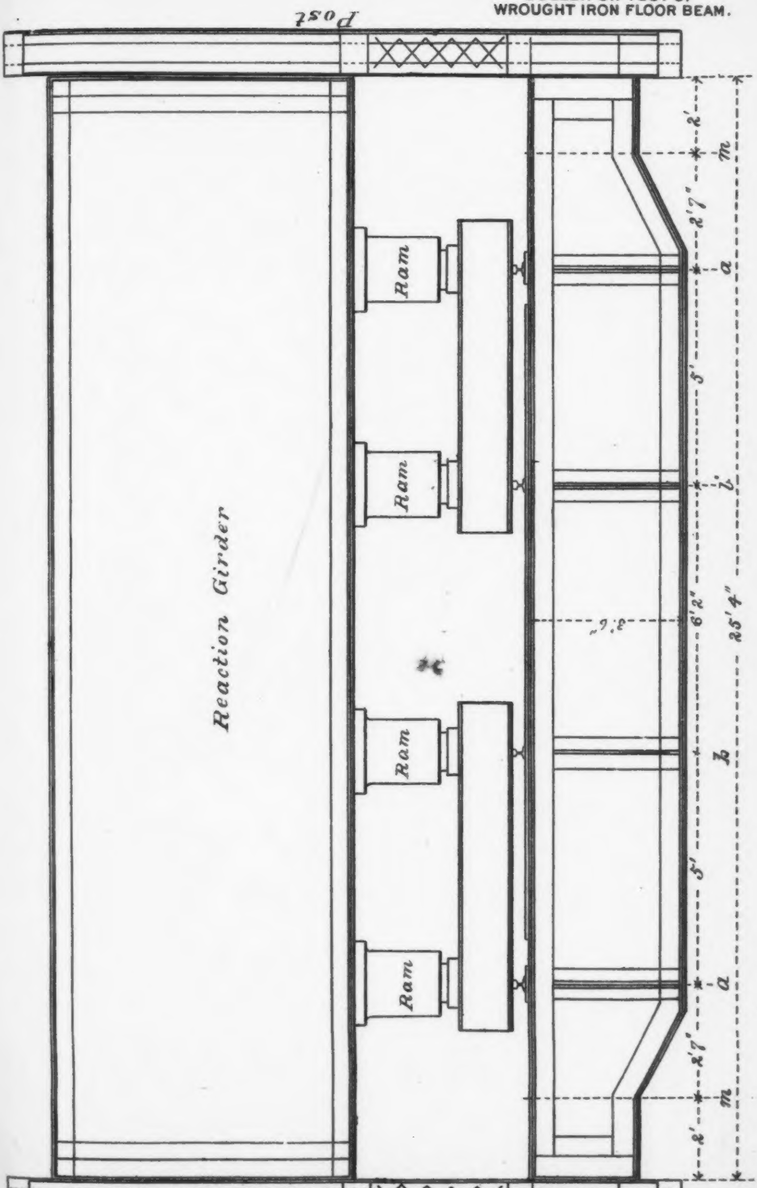
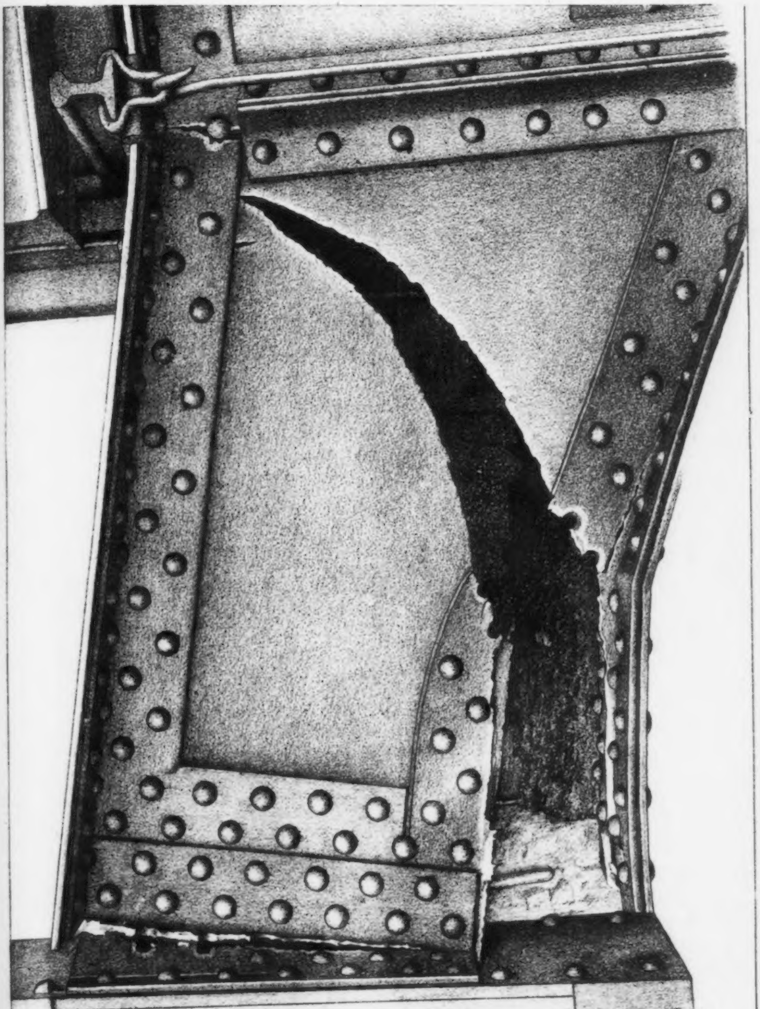


PLATE XXXVIII
TRANS. AM. SOC. CIV. ENGRS.
VOL. XVIII Nº 382.
BOLLER ON TEST OF
WROUGHT IRON FLOOR BEAM.



DISCUSSION.

George Devin, M. Am. Soc. C. E.—Reviewing the subject discussed in the interesting paper so kindly submitted to the Society by Mr. Boller, I find that the results are entirely in harmony with well understood and accepted principles in mechanics and that the strength of materials developed in the test coincides with the results prescribed in our every-day engineering practice.

I would quote the usual specification for the strength of material under the various conditions here brought into play:

	Pounds per Square Inch.
Ultimate strength in the direction of the grain of plate and angle	45 000 to 48 000
Strength across the grain	32 000 to 40 000
The crushing strength of wrought-iron in short lengths or in partly confined sections applicable to rivet bearings	60 000 to 70 000
Shearing strength of plate iron	40 000
Shearing strength of iron rivets	40 000 to 42 000

Where diagram and original data is called for in this article, reference is made to the original drawing (Plates XXXVI and XXXVII) and the data given in Mr. Boller's paper.

It is the usual practice in proportioning plate girders to assume that the bending or chord stresses are resisted entirely by the flanges and the shearing or web stresses as resisted entirely by the web. Strictly speaking, this assumption is not true. The result of our proportions as usually made being a slight surplus in the material available for flange duty. Accurately considered, one-sixth of the web is available for and does flange duty. Wishing to examine as accurately as possible the conditions of stress in the girder under consideration, we will take into account the resistance to bending available in the web.

Since in built girders formed of plates and angles, as in this case, the only stress on the flange comes through the connection of the web to the flange, we may properly take the efficiency of the connection as the available flange section (assuming that the flange is equal to or in excess of the connection).

Referring to the data given in the experiment, we have as the end reaction at the instant of failure, 326 000 pounds, and the resulting horizontal or chord stress at the point $m = 284\,000$ pounds to be resisted by the rivet connection plus so much of the web as is available for flange duty. That is, the bearing section of ten $\frac{15}{16}$-inch rivets on $\frac{3}{8}$-inch plate $= 3.5$ square inch plus one-sixth of the web plate whose net section is 26 inches $\times$ $\frac{3}{8}$-inch $= 1.6$ square inches, giving us a total section of a little more than 5 square inches. Then we have as the stress imposed per unit on the available section $\frac{284\,000}{5}$ pounds $= 56\,800$ per square inch.

Under the foregoing assumption, this then is the stress imposed alike on the rivet-bearing and on the web section available for flange duty. This is true for the compression flange when the bearing capacity of the rivet and the web plate is the same.

Considering now the tension flange or the lower chord at the point *m*, we have from our previous deduction the section of the flange material proper made available for duty by means of the strength of the rivet connection, 3½ square inches out of a total section at that point of 18½ square inches.

Draw a line from the bottom edge of the girder at the end, to the bend near the point *a*, to represent the direction which the line of stress in this lower flange endeavors to take between these two points.

Through the reverse bend at the point *m*, draw a line perpendicular to this line, and call this the normal to the assumed line of stress. Knowing the horizontal stress in the flange between the end and the point *m*, we deduce the stress in the direction of the normal which is developed by the horizontal stress. By calculation we determine the normal stress in this case to be 150 000 pounds. Now, this stress is theoretically applied at the point of the bend. Practically from the shape of the flange at this point it must initially be distributed over a very short distance. Let us assume that it may be distributed over as much as 4 inches of distance along the flange.

The material to resist this normal stress is mainly the section through the vertical leg of the 6 inches by 6 feet by ½-inch angles of the angle at the root—this would give us as the resisting section 4 inches by 1-inch across the grain of the material to resist 150 000 pounds, or a stress of 37 500 pounds per square inch, the ultimate strength of material under such conditions.

Assuming that this stress commences to part the vertical from the horizontal leg of the angle, we see that at the instant the part commences, we have at this point only the section of the vertical leg of the angle remaining as flange material proper, whose net section is about 5 square inches.

Now, at this point the material being under tension, and the bearing strength of the rivet connection being superior to the tensile strength of the material, we shall have an increase of stress on the bearing section of the rivets until the tension stress is uniformly distributed over all the material available for tension, or until failure occurs in the bearings.

In other words, we have as the material available for tension the material in the vertical leg of the flange angles = 5 square inches plus one-sixth of the web section = 1½ square inches; total, 6½ square inches to resist the horizontal stress at this point, 284 000 pounds, or 43 700 pounds per square inch, which is a fair average ultimate strength of plate and angle in the direction of the grain.

The bearing stress on the rivets in the stress transmitted to the 5 square inches of section in the vertical leg of the angles = 43 700 pounds × 5 = 218 500 pounds on 3½ square inches of bearing, or 62 430 pounds per square inch, which is about the usual resistance developed under such conditions. Briefly reviewing, we find that only a small portion of the flange section at the point of rupture (5 square inches out of 17 square inches net section) was utilized in resisting flange stress, and that the balance was lost from use on account of the direction given to the flange in the design. We can only say that the stress on the web plate and rivet connection was close to the breaking limit as determined by such experiments as have been made on full-sized built girders and on detail pieces.

Albert Lucius, M. Am. Soc. C. E.—The fact that in this particular experiment the chord angles gave way at a comparatively low strain, and that the web and the riveting stood comparatively such high strain, has been made more of than is warranted by this very inconclusive experiment. The floor beam was acknowledged to be of bad shape, and under test it failed at the bend in the lower chord, near the end of the beam, where it was most likely that it would fail, and it stood between three and four times as much load as it was exposed to in service. The chord angles gave way primarily by the horizontal flange shearing off from the vertical flange, and the latter and the web then broke. This shearing was due partly to the shape of the girder and the sharp bend and offset so near the end, and partly it may have been due to a flaw induced by the bending, or originally in the angles. The experiment sheds no light on this question. The high resistance to shearing and buckling in a shallow and stout web, and the high resistance of riveting especially done for a test-piece are also results not to be wondered at.

Charles O. Brown, M. Am. Soc. C. E.—The form of floor beam under discussion is one that bridge engineers dislike, and consequently try to avoid. It was adopted in this case probably for the purpose of obtaining increased central depth without encroaching upon the waterway. Any person at all accustomed to looking at ironwork would point out the section at *m* (using the designations upon Mr. Boller's diagram) as the weak place. Here the lower flange is bent and the strain being obliged to follow the bend, cannot do so without producing a down pull upon the web and a tendency to separate the horizontal flange of the angle from the vertical flange, and this at a point where the structure of the angle iron has been weakened by heating and bending. It is a well-known fact that bending of large size angles will frequently cause such separation.

With a new bridge, of recent construction, there should be no discussion as to whether it will bear the traffic imposed. The existing standard specifications allow an ample margin for safety, and the question in this case is simply whether the design of these floor beams

complies with such standard specifications. There is no need of any test; analysis of strains will show it.

Taking Mr. Boller's figures (without adding for impact—allowing that to be taken care of by the low-working stresses fixed in specifications) the shearing strain in end section of web plates is 8 000 pounds per square inch. This would be twice the working strain allowed by standard specifications. The actual strain per square inch, after deducting rivet holes is 9 500 pounds or 2.4 times the working stress. The chord strains to be transmitted from *m* through ten rivets is 80 000 pounds or 8 000 pounds per rivet, this again is twice the working stress. These are two points of only one-half the strength that a good structure should have. The beam, if tested, should show a factor of safety of seven, or it should stand before failure a test load of 1 120 000 pounds. It did resist, according to record of test, a little over one-half that load, when it failed at a place which theory and experience points out as the weakest.

Mr. Boller's deductions from the experiment are:

First.—That a plate web is an exceedingly stiff member.

Second.—That our method of proportioning rivets is wrong.

Third.—That the friction due to power riveting has enormous value.

These statements, if intended to allay the fears of the railroad officials or the public, with reference to that "unsafe floor beam," will pass; but when they are brought before the Society, I wish to enter my protest. The friction is too uncertain a quantity to be relied upon, and the experiment certainly shows nothing that would warrant an increase of working strains on web plates, or of the bearing pressure of rivets.

The whole discussion shows conclusively that the knowledge on the subject is sufficiently thorough, and that our standard specifications are safe. So that by the exercise of due care and vigilance in designing and manufacturing we can avoid "unsafe floor beams," and unreliable structures in general.

Theodore Cooper, M. Am. Soc. C. E.—The author having, to his own satisfaction, proved that the accepted practice in designing plate girders is all wrong, it may be well for those whose experience and observation in this field have led them to other conclusions, to protest against so broad a claim being based upon a crude test of a single girder, even should an analysis of this single test hold out the claims.

There is hardly a single fallacy advanced by the ignorant pretenders in the science of engineering that has not been supported by evidence even stronger than that presented in this case.

The application of our accepted rules for designing girders does not show this girder to be a dangerous one under the proposed testing. It does, however, show that the girder is a badly proportioned one.

Does this test prove anything more than this?

The true measure of the strength of a girder is the strength of its

middle section, and a properly designed girder should have its details so proportioned that this maximum strength can be realized.

This girder, as shown by the author, gave way by rupture of the ends before the center flange areas were strained to one-half their full strength, showing very clearly that the ends which should have been the stronger, had only half the needed strength.

I think, therefore, we may conclude from this test, that better results would have been obtained if the "accepted practice in designing plate girders" had been applied to the design of this one. Let us now examine whether the test has proved in any manner that our theories are wrong.

In proportioning plate girders we now determine the number of rivets to be used, upon the basis of the theoretical strains in each part, with certain allowances for shearing strain on the rivets and for the bearing or crushing area of the web plates.

The test shows that the rivets have not sheared nor the webs been torn out in the manner called for by the theory. But we must take other things into consideration. It is well known that the frictional resistance of the rivet heads is very large. Mr. Considère found by a series of experiments that before there was any slipping of plates riveted together this frictional resistance amounted, on an average, to 18 000 pounds per square inch of rivet area, and that it amounted to about 42 000 pounds per square inch of rivet area before actual rupture.

English and American practice has not allowed for such frictional resistance in proportioning the rivets, as there was no certainty how long it would continue after structures were subject to actual service. The general practice of depending upon the shearing resistance and bearing areas and not on this uncertain frictional resistance, has seemed more justifiable.

This girder, made especially for test, was not likely to have any defective workmanship in its construction. The frictional resistance of the newly-driven rivets was undoubtedly high and prevented the rivets acting either by shear or crushing.

Does a girder like this, fresh from the workmen's hands, represent the condition of a similar girder which has been hammered over for years by heavy and rapidly moving trains, with all the conditions of good, bad and indifferent tracks ?

I should regret to find that any one had accepted the conclusions of the author that "all formulas for proportioning plate girders are utterly in the fog," and "the dicta of one expert is as good as that of another."

Rule of the thumb designing may be good enough for those who are loath to advance beyond their early practice.

The advances that have been made in recent years in bridge and girder designing have been due to the better consideration of the pro-

portioning of the details and a crude test of this kind will hardly be sufficient to convince us that we are all wrong and "in a fog."

WILLIAM H. BURR, M. Am. Soc. C. E.—A positive determination of the causes of failure of this beam at such an apparently low flange stress at *m*, involves a much clearer knowledge of the action of internal stresses in a plate girder than is at present possessed. If the ordinary methods of plate girder computation be assumed, however, as does Mr. Boller, some very significant results are obtained.

At the instant of failure the flange stress at *m* was 12 600 pounds per square inch (presumed to be on total area). The total sectional area of the two 6 x ½-inch horizontal angle legs at the 14 x ½-inch cover is 19 square inches, which, at 12 600 pounds gives a force of 239 400 pounds. This force, it is to be remembered, does not pass directly into the inclined portion of the flange. As that inclined portion makes an angle to a horizontal line whose sine is 0.537, the force 239 400 pounds has a component normal to the inclined portion of the flange equal to 239 400 $\times$ 0.537 = 128 558 pounds. This, it is to be observed, is a force pulling normal to the fibers of the flange angles in the section *m*, and is concentrated at the bend. If it be assumed that it is distributed over 3 inches in length of the two ½-inch thick vertical legs, this normal force will produce a stress per square inch across the fiber of the angle of 128 558 $\div$ 3 = 42 852 pounds, which is about sufficient to produce exactly the amount and kind of failure that took place. These figures, of course, have the approximate character which all such computations possess, but no more. They show just why the kind of failure observed took place, although the quantitative determination could not accurately have been made in advance.

In connection with questions of this kind it is seldom or never remembered that every rule for flexure and long column comparison used by the engineer in his daily practice is only loosely approximate, and, strictly speaking, not properly applicable. They can only be correctly applied to beams or columns whose lengths are exceedingly great in comparison with either of their cross dimensions; as they are, and can only be established in the theory of elasticity in solid bodies by making just those assumptions. These rules and formulas, of course, form the basis of all structural computations after empirical quantities, determined by such experiments and tests as that under consideration, are introduced in them, but are essentially valueless for practical purposes until that is done. The author is not far wrong, therefore, in stating that theory applied to such beams leads to very foggy results. Nor is it easy to overestimate the value of such tests as this, for by such means only can the "fog" be dissipated.

After a careful consideration of the results of this test, I cannot find the evidence of the rivet friction which the author of the paper and others believe they find.

Alfred P. Boller, M. Am. Soc. C. E.—While admitting the force of the various comments made upon this experimental girder, which we all agree was of a very bad shape, due to the bend in the lower flange near the ends, all criticisms that had been passed upon it, selected the end web with the connecting rivets as the weakest part, thus determining the strength of the beam. This on theoretical considerations. The fact that the beam failed entirely in the flange under a computed strain (reaction, by distance, divided by depth) comparatively low, before the web was affected with its connecting rivets, is hardly an harmonious agreement between theory and practice, and while perhaps characterizing beam formulas as being "all in a fog" is rather an extreme statement, it would seem in this particular case, the result of the experiment showed a wide gap between theoretical prophesy and actual fact. Nor will it do to brush the experiment to one side as being "crude" and of no importance. It was an actual beam destroyed under pressure, under as fair conditions as it is possible to have—pressures applied by hydraulic power furnished from the testing apparatus of one of our greatest bridge works—and while one may quarrel with gauge readings and absolute values, the relations of the behavior of the several parts of the girder are very clearly recorded. Mr. Devin very ingeniously shows a computation that fairly well fits the experiment after the angle irons had split along the "root," when the tension flange became practically of very small area; but what mode of computation, or what theory could forecast the behavior of that beam had the flange held solid beyond an approximation based upon assumptions. The theory of beams and their flexure is probably as perfect as it ever can be, but, as applied to practice, so many physical and constructive elements come in, that results at best can be but approximations, never exact, involving the exercise of sound judgment quite as much as refined analysis. There is no question, that generally the modern methods of designing and detailing girder work are a vast improvement over even comparatively recent practice, and are all that can be desired so far as safety is concerned, but we must not forget that such work, in the nature of things, is far from theoretical perfection, and should not be used as a basis of condemnation of earlier executed work without very thoughtful consideration. Were theory perfectly applied in practice, there should not be much difference of opinion among expert engineers. But there are many questions that must be settled on pure judgment, and here the dictum of one expert may be just as sound as that of another. Too great a devotion to figures may insensibly affect the judgment, by accustoming the mind to a dogmatic confidence in theoretical conclusions, which, while possibly well enough in new work, becomes a serious matter when applied to work of the past—as fully illustrated by the test and comments on this beam.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

383.

(Vol. XVIII.—May, 1888.)

SYPHONS OF THE KANSAS CITY WATER WORKS.*

By G. W. Pearsons, M. Am. Soc. C. E.

Presented for Annual Convention of 1888.

WITH DISCUSSION.

The first water supply for the Kansas City Water Works was drawn from the Kansas River, one and-a-half miles from its junction with the Missouri; at this place the state line is 700 feet east of the river, the proposed location of the works being in Kansas.

The Water Commissioners fearing complication from this, desired the pumping station to be placed in Missouri if practicable. The nearest available location where drainage could be had for the settling basins was a quarter of a mile, and the conditions such that a syphon was the only available means of reaching the pumping station. The soil is a sandy alluvium, surface 28 feet above low water; about half of the depth can be excavated readily; the lower part, when dug into, shows decided characteristics of quick sand, making a deep trench both difficult and dangerous.

A trench low enough to insure adequate flow at low water was impracticable; a syphon to be safe, must be tight, and to insure this, it must be reasonably accessible.

* These were shown in a description of the Kansas City Water Works, in "Engineering News," November, 1887. The writer has been requested to give a fuller description.

The trench was dug 13 feet deep at the pumping station, 15 feet near the river, descending by an easy vertical curve; the pipe, 24-inch cast-iron, with ordinary lead joints, was laid with care as to grade, so that no air pockets should exist, rising all the way to the well, where it joins a vertical pipe by a tee, with well-rounded angles; the top of this vertical pipe rises 6 feet above the syphon, forming a vacuum chamber; the bottom reaches within 18 inches of the bottom of the well, which is 7 feet below low water in the river, giving an available slope of 5 feet at extreme low water; there are no bends in the line, except such easy curves as could be laid with straight pipe; its delivery is ten millions of gallons in twenty-four hours, with a frictional resistance of 22 feet per mile, or 4.16 feet per thousand.

During high water, the syphon being practically submerged, acts as a conduit, and is controlled by a valve at the pumping station. It was carefully tested as to tightness before filling the trench, but it was found necessary to maintain the vacuum by an air pump attached to one of the engines. This was 3-inch bore, 12-inch stroke, making 18 to 24 strokes per minute, sufficient at first, but within a year it became necessary to aid it by a pipe to one of the condensers. After some five or six years run the air leakage made a reduction in the vacuum in the condenser; the pipe was then dug down to and the joints recaulked, recovering its original tightness; a larger air pump was also put in. In the twelve years of its run, the syphon has never failed, and the only expense attending its use, aside from the working of the air pump, which being attached to the machinery makes no appreciable labor or cost of maintainance, is the recaulking of the joints, which was expensive on account of the depth of the digging required to reach them.

At Quindaro, the new pumping station on the Missouri, it again became desirable to go away from the river for safety. There the syphon is 730 feet long, 42 inches in diameter, cast-iron, lead joints, bottom of pipe trench so soft that it had to be planked to preserve the alignment of the pipe. A steam ejector is used to maintain the vacuum. At present the pipe is so tight that it requires but a few minutes use daily, a vacuum guage being placed in the engine room for the engineers guidance. Rise of syphon (center) 10 feet. The pumping engines at this station have a capacity of one million gallons per hour, not enough to test this syphon to anything like its full capacity, which is probably over 50 millions per day—having an available slope of 12 feet per thousand.

The use of the first syphon showed that while absolutely air tight there was still a sensible amount of air constantly removed by the air pump, showing that it was dissociated from the water; the writer had no means of making quantitative tests, but from observation saw that the amount of air varied sensibly in accordance with the amount of water flowing through the pipe, and that with the full lift of 16 feet the dissociation was much greater than with less lifts—seemingly greater than the difference.

The use of these syphons show that they can be safely depended on in places where it is difficult to lay conduits low enough for direct gravity flow, and that the receiving well which they require is a safe-guard to the syphon in avoiding pulsation or shocks, which would tend to produce air leaks in the joints, and insures the pumps against unequal or undesirable admission of air as well as against the water ram incident to a long suction pipe.

DISCUSSION.

A MEMBER.—I want to ask Mr. Pearsons whether the removal of the air from the water makes the water unhealthy?

MR. PEARSONS.—It does not appear to me that there is a sufficient amount of air withdrawn from the water in its passage through a quarter or an eight of a mile of vacuum line to make any difference. Besides the air thus withdrawn would be very soon restored in the storage basin.

I would say, with regard to aeration, that when the water was passing under the ice of the Missouri River, aeration was beneficial, but that after the river was entirely open and free from ice, I could not find that aeration did any further good.

C. B. BRUSH, M. Am. Soc. C. E.—Was there any smell or taste developed in that water during the summer?

MR. PEARSONS.—In the upper reservoir, where the water stood still for a long time, there was something of that sort. In our lower basin we do not have anything of the kind, because the water stands in it but a few days.

MR. BRUSH.—For how long a time was this storage?

MR. PEARSONS.—About a week in the lower basin.

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(Vol. XVIII.—June, 1888.)

THE VENTURI WATER METER.*

DISCUSSION ON PAPER No. 371.

By Clemens Herschel, M. Am. Soc. C. E.

Since the experiments of 1887, and since October and December, 1887, when the original paper upon the "Venturi Water Meter" was written for and read before the Society, the meter has made rapid progress. At the present time it constitutes the introduction of a new and pregnant principle into the art of gauging fluids, such as water and other liquids, also steam, compressed air, fuel or illuminating gas, and other gases. The formation of any degree of vacuum, or negative pressure at the throat of the instrument, which first called attention to this method of gauging, has become only a special case of its action, while the measurement of the difference of pressure between the fluid in the main pipe, just above the meter, and as it passes the throat, has been found to constitute a precise measure of the linear velocity, hence of the volume, passing through the throat and through the main pipe.

Difference-Pressure Gauge.

In the discussion immediately following the reading of the paper, it was asked, whether the record of this difference of pressure could be kept automatically, and following the suggestion thus thrown out, I have contrived the difference-pressure recording gauge herewith shown. It is believed that this is the first gauge of this sort ever constructed,

* The Venturi Water Meter. By Clemens Herschel. Transactions, Vol. XVII, p. 228, November, 1887.

barring some attempts to photograph the indications of the two legs of a Pitot tube used in measuring current velocities, and thus to obtain a continuous picture, or record, of the relative positions of the liquid within the Pitot tubes.

A very few words will explain the action of the gauge (Plate XXXIX). P_1 and P_2 are the two piezometer tubes, the one leading out of the main pipe, just up-stream from the meter, the other leading out of the venturi air-chamber. These are conducted to any convenient point, where is set up the difference-pressure gauge. This swings like a pendulum in stuffing-box bearings, as shown in cross-section Figs. 2 and 3. A diaphragm separates the two pressure conduits, so that the mercury columns, M_1 and M_2 are respectively acted on through connection pieces, A_1 and A_2, by the pressure in P_1 and P_2. To give more swing to the pendulum, the ball, B_1, is the heaviest, and will set the pointer at 0 of the arc when the two mercury columns are on a level. As soon as any appreciable difference of level is obtained between them, however, due to a difference of pressure in P_1 and P_2, the gauge will swing away from the 0 of the arc, which can thus be made to indicate directly cubic feet per second or per hour, or any such quantity, passing the meter. The mercury tubes are shown as though made of glass; but this is by no means necessary, as iron or steel will answer equally well. This removes one usual objection to ordinary mercury columns. Another one, that of spilling the mercury by water or steam ram, will not occur in this form of apparatus, I believe: because shocks of this sort, passing the meter, seem to affect both pipes, P_1 and P_2, simultaneously.

I have tried only one meter as yet under high heads, with an ordinary steam pressure gauge set on P_1, and another on P_2, and have the belief above expressed in the light of the only experience thus far acquired. That a gauge of this kind can be made to keep its own automatic record need hardly be stated.

Other forms of difference-pressure gauge can, and will no doubt be devised; and upon the construction of very sensitive gauges of this sort depends the perfection of the meter; whose only admitted defect, at present, is its inability to meter very small quantities. Or, in other and more precise words, whose range of capacity is somewhat limited.

A Worthington $\frac{5}{8}$-inch meter, tested in California (Ross E. Browne's Experiments, Van Nostrand's Science Series No. 81), had a range from $\frac{1}{15}$ to 7.5 gallons per minute, or from 1 to 112.5.

A Crown $\frac{1}{2}$-inch meter (tested in New York, 1879) had a range from $\frac{1}{26}$ to about 13. gallons per minute, or from 1 to 338 (nearly). If the least difference of pressure in P_1 and P_2, above referred to, that can now be measured, be taken at $\frac{1}{4}$ pound per square inch, it would make a range, for the 1-inch meter, spoken of later on, from 1.47 to 31. gallons per minute, or from 1 to 21. If this lowest measure of pressure differences be assumed, however, at $\frac{1}{10}$ of a foot, as was actually done in Ross E.

Difference Pressure Recording Gauge,
to Accompany
Discussion of Paper No. 371.
"The Venturi Water Meter."
by Clemens Herschel,
M. Am. Soc. C.E.

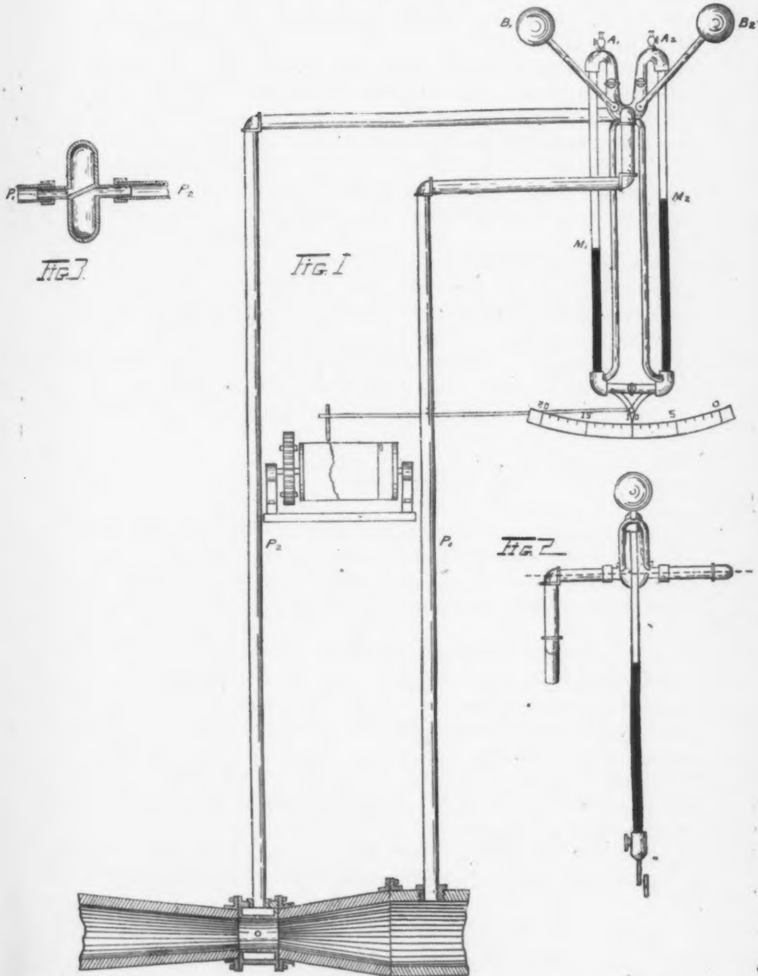

PLATE XXXIX.

Browne's water meter experiments, the above range of capacity would be increased from 1 to 21 to something greater.

And all this only shows that the Venturi meter, from not having been designed for domestic service, does not meet this and other requirements of that particular service. On the other hand, to meter the quantity supplied to large establishments, to a whole district of a city, or to the whole city itself, though that city be of the size of the City of New York, is something that can be done with great ease by the Venturi meter; and it has for such purposes ample range, accuracy and durability, cannot clog up, and keeps well within the practical limits of what such a meter may cost. Other uses of the meter, under low heads, would arise in connection with irrigation works, filtering plants, to meter wash water in industrial works, and water used for power purposes. The meter is also applicable to the gauging of steam, compressed air and other gases.

Generally speaking, a 6 or 8-inch supply pipe limits the size of ordinary water meters, and very few are built, or used, having so much as four inches inlet and outlet; while the Venturi meter is probably unlimited in the size at which it may practically be built and operated, ranging from capillary tubes, to tunnels; but does not meet some of the other requirements demanded of those water meters which are to be used for supplying dwelling houses and other domestic establishments.

Tests of a One-Inch Meter.

Since paper No. 371 was read before the Society, I have tested a 1-inch meter of the sort therein described, under 210 feet head. The interior of the meter was in outline precisely similar to the other two, already described, but the meter itself was a brass casting, cut to shape and finely polished inside.

Unfortunately there were no facilities at hand to measure pressures accurately. The pressure gauges used were common, cheap, steam-pressure gauges, indicating possibly five or more pounds variance from the truth. Were it not for the fact that their differences alone were used, and that they could always be compared among themselves when the water was still, the results would have been entirely worthless. I submit the table of results which follows, with great diffidence, and only as being better than nothing at all. By reference to what was said concerning the tables given for the 1-foot and for the 9-foot meter, the following table for the 1-inch meter will be readily understood.

TABLE No. 1.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
No. of experiment.	Duration of experiment.	Quantity of water passing venturi.	Velocity through venturi.	Head Gauge.		H = Total Head.		Co-eff't $= \frac{\text{Velocity}}{\sqrt{2\,g\,H}}$	Depression gauge.	Working of pet cock.	H_v = Head on venturi.		Co-eff't $= \frac{\text{Velocity}}{\sqrt{2\,g\,H_v}}$	Theoretical $H_p = H'_v$	Theoretical co-efficient.
				Upper.	Lower.										
	min.	Cub. ft. per hr.	Feet per second.	pounds	per	square inch.	Feet.		pounds per sq. in		pounds per sq. in	Feet.		Feet.	
1	7.5	49.5	21.5	88.	87.7	0.3	0.69	3.23	86.	Runs Water	2.	4.62	1.25	4.72	1.23
2	7.	80.1	34.8	86.	85.2	0.8	1.85	3.19	80.	"	6.	13.86	1.16	14.12	1.15
3	7.	100.5	43.6	85.	83.7	1.3	3.00	3.14	75.	"	10.	23.09	1.13	23.50	1.12
4	7.	117.9	51.2	84.5	82.6	1.9	4.39	3.05	70.	"	14.5	33.49	1.10	34.05	1.09
5	6.5	142.8	62.0	82.	79.1	2.9	6.70	2.99	60.	"	22.	50.81	1.08	51.63	1.08
6	7.	170.3	73.9	81.	76.9	4.1	9.47	3.00	49.5	"	31.5	72.75	1.08	73.92	1.07
7	7.	190.0	82.5	79.	74.0	5.0	11.55	3.03	40.	"	39.	90.07	1.08	91.53	1.07
8	5.	210.7	91.5	78.	71.7	6.3	14.55	2.99	30.	"	48.	110.86	1.08	112.65	1.07
9	5.	227.5	98.8	76.	68.8	7.2	16.63	3.02	20.	"	56.	129.32	1.08	131.41	1.07
10	5.	239.4	103.9	73.	65.0	8.0	18.48	3.01	11.	"	62.	143.19	1.08	145.50	1.07
11	5.	254.4	110.4	72.5	58.5 to 0	14. to 72.5	32.33 to 167.44	2.42 to 1.06	2.5	"	70.	161.66	1.08	164.27	1.07

It will be observed that no vacuum could be formed in this meter when starting with 88 pounds pressure. But by reducing the head upon the meter, a vacuum just commenced to form, when the other pressures were as given in the following table:

TABLE NO. 2.

PRESSURE AT HEAD GAUGE.	PRESSURE AT VENTURI GAUGE.	PRESSURE AT A GAUGE DOWN-STREAM FROM THE METER.
In Pounds per sq. in.	In Pounds per sq. in.	In Pounds per sq. in.
51	0	38
41	0	34
31	0	25
21	0	15
11	0	6

PROPER SHAPE FOR THE METER.

The subject of the proper shape for converging and diverging tubes has long occupied the attention of scientists and of mechanics, but owing to lack of experimental data, is yet unsettled. It has been lately treated in a paper by A. F. Nagel, C. E., before the 1888 meeting of the American Society of Mechanical Engineers.

According to the belief therein expressed, at which the author seems to have arrived independently of its suggestion previously by others, the simple rule to be followed is, that the acceleration, or retardation, of the fluid must be uniform, while passing through the converging or diverging tube; being the same idea which was suggested to me in 1874. This produces one form of trumpet-shaped tubes, but leaves the proper length of the tubes undetermined. Least loss of head while passing the meter, must be advantageous, and this would call for as short a length as possible. It is evident that experiment alone can answer some of the questions that arise in this connection. A Venturi meter formed by joining two trumpet-shaped or conical tubes of equal dimensions, at their smaller diameter, would permit of its being used to gauge liquids flowing in either direction, which has a practical value in certain locations; and a refined form of interior lines, should have practical value, in the cases of high heads more especially.

PROPER MODE OF ATTACHMENT, TO THE METER, OF THE PIEZOMETER TUBES.

This has always been a puzzling question to hydraulicians, and the results of many experiments are no doubt worthless or unreliable,

from no other cause than this of faulty attachment or arrangement of piezometers.

The writer means to try whether the mode of attachment originally adopted for the tube leading from the Venturi to its gauge, be not also the proper mode of attachment for the tube leading from any part of the Venturi meter to its appurtenant gauge, and believes that such mode of attachment will give the best results.

Volumetric Measurements.

In the public mind, when the word water meter is mentioned, there immediately arises the conception of a little cast-iron box, with a set of dials on top, on which may be read off the cubic feet or gallons of water that have passed the meter since the dials last stood at zero. This constitutes a volumetric meter, or integrating meter, as it has been called.

Essentially different from this class are so-called differential meters (carefully to be distinguished from inferential meters, which are only a species of volumetric or integrating meters). Of the differential class, being meters which indicate and measure rates of flow, there are, as yet, in this country but two kinds—the Deacon waste-water meter and the Venturi meter. Considering now the latter, if it be fitted with any of the ordinary pressure gauges only, it will serve to indicate, by means of a simple computation to be made by the observer, what is the rate of flow through the meter at any moment of observation.

Second.—If fitted with the difference-pressure gauge above described, it would indicate directly, on the circular arc of the gauge, the rate of flow in cubic feet, or gallons, per second, or other unit of time, at the moment of observation.

Third.—If provided with a self-recording apparatus, it would keep, automatically and continuously, a record of such rate of flow, and would thus furnish the data from which the total volume that had passed the meter between any two intervals of time could be computed.

Fourth.—A further step, and one that would convert the differential into an integrating meter, would be to cause the computation last named to be continuously performed and indicated on a train of dials by suitable mechanism, such mechanism being called "mechanical integrators," and many forms of which are in existence. The writer has himself devised one such form, and a very simple one, for the case in hand, and while he has no immediate expectations that a small volumetric meter constructed in this manner can compete with the several simple forms of ordinary small water-meter, yet it is none the less interesting to note that the ordinary volumetric measurement of quantities up to 150 million gallons per twenty-four hours, and even more, is now easily within the compass of a single meter with an accuracy of probably less than $\frac{1}{2}$ of 1 per cent, and at a comparatively very small cost.

Similarly for smaller volumes, from those flowing in 1-inch pipes, or smaller ones, upwards.

The same instrument is also readily arranged to do both kinds of service; to keep a continuous record of the rate of flow, and to indicate on a train of dials the continuous summation of the volumes that have been passing the meter.

DISCUSSION.

Mansfield Merriman, M. Am. Soc. C. E.—The slight range in the co-efficient, determined by the experiments of Mr. Herschel, show that the Venturi meter gives promise of furnishing a precise method for the measurement of water. It appears to be a form of meter which can not only be used for gauging the flow in mains, but can also be advantageously employed in tests of water motors instead of a weir, particularly in cases where a large number of measurements are to be made. It seems desirable that such a form of tube should be adopted as to give the least loss of head, and the shape used by Mr. Herschell is that which has generally been regarded as giving the maximum discharge, being similar to that regarded as best by Venturi and other experimenters. The tests made by Mr. Francis in 1854 showed that with such a form of tube a maximum discharge of about two and one-half times the theoretic discharge could be obtained, and as no experiments have given a greater ratio it might be inferred that this is the form which gives the least loss of head. In a recent paper before the American Society of Mechanical Engineers, Mr. A. F. Nagle has proposed a form for nozzles and compound tubes which appears theoretically to be more perfect, its sections varying in such a manner that the acceleration is constant. This requirement seems to be the correct one, and can be justified in a number of ways, the simplest being that of analogy from the sections of a jet which rises vertically from an orifice, and attains to the level of the surface of the reservoir. In this case no head is lost, and the acceleration is uniformly decreased, so that a perfect nozzle may be regarded as one whose sections vary in accordance with the same law, frictional resistance being disregarded. To deduce the law of variation of sections let h be the height of the jet and x be any distance above the orifice, and let d and y be the diameter at the orifice and at the height x; the corresponding velocities are $\sqrt{2\,g\,h}$ and $\sqrt{2\,g\,(h-x)}$, and since the velocities are inversely as the areas of the sections,

$$\frac{y^4}{d^4} = \frac{\sqrt{2\,g\,h}}{\sqrt{2\,g\,(h-x)}}$$

from which the diameter y is

$$y = d\left(\frac{h}{h-x}\right)^{\frac{1}{4}}$$

This is the equation for a perfect jet, in which y becomes infinity when x equals h. Now to apply this to the deduction of a form of perfect nozzle let D be the diameter of the large end, d that of the small and l its length. Then y is replaced by D and x by l, or

$$D = d\left(\frac{h}{h-l}\right)^{\frac{1}{4}}$$

and eliminating h from the two equations there is found

$$y = d\left(1 - \frac{x}{l} + \frac{x}{l}\frac{d^4}{D^4}\right)^{\frac{1}{4}}$$

From this formula values of y may be computed for any assumed values of D, d, l and x, giving a form of curve similar to that shown in the figure. Two of these joined together at the throat may be taken as the Venturi tube of maximum discharge or least loss of head. Theory offers no hint as to the best relations between D, d and l, but the length of the part below the throat should probably be two or three times that above it.

The following table gives the result of a few computations on the losses of head which occurred in the large meter experimented upon by Mr. Herschel (Transactions, Vol. XVII, page 250). Here P_1 refers to the upper piezometer, P_2 to that at the throat, and P_3 to the lower one. The loss of head given in the third column was directly measured by the difference of the readings of the piezometer P_1 and P_3; the other columns have been computed from the given velocities and diameters.

1. No. of Experiment.	2. Velocity in feet per second.	3. Loss of head between P_1 and P_3.	4. Loss of head between P_1 and P_2.	5. Loss of head between P_2 and P_3.	6. Ratio of col. 5 to col. 4.
1	1.75	0.005	−0.008	0.013	
4	4.96	0.045	+0.007	0.038	5.4
5	7.64	0.102	0.024	0.079	3.3
9	10.63	0.175	0.086	0.089	1.0
17	22.98	0.905	0.668	0.236	0.36
27	29.32	1.493	1.263	0.229	0.18
43	32.98	1.869	1.617	0.252	0.16
47	34.47	2.049	1.873	0.176	0.09

This table shows that where the velocity was low the loss of head mostly occurred below the throat, while the reverse was the case for high velocities. It may be thought that in the latter event the stream, after passing the throat, did not touch the sides of the tube at all, but that it expanded according to the law of nature, its sections increasing as its acceleration uniformly diminished. It would be a matter of much

scientific interest if experiments could be made upon the form of tube which theory indicates as the most perfect.

JAMES B. FRANCIS, Past President Am. Soc. C. E.—The theory of the Venturi tube has been known for a long while, but there have been few practical applications of it. It is said that the Romans knew how to get more water into the pipes in their houses by putting a Venturi tube at the end of the pipe. It is this same principle which Mr. Boyden applied in his diffuser to increase the efficiency of the turbine water wheel, but this is a much wider application and it is entirely new, so far as I know. I expect it will overthrow some of our theories of water flow, though whether all of them, is a question. It remains to be proved by experience what changes may come from the use of this tube; still, I think, it is an entirely new application of the old principle.

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TEST OF AN EDISON INCANDESCENT ELECTRIC LIGHTING PLANT.

By JOHN W. HILL, M. Am. Soc. C. E.

READ MARCH 7TH, 1888.

WITH DISCUSSION.

The writer presumes that all members of the Society are interested in the electric light, if not by direct contact with it as constructors or users, at least from the stand point of speculation as to its actual efficiency, comparative cost and general application; and that any practical information (however meager) in this field, will be acceptable for what it is worth.

The writer pretends to no other knowledge of the subject than that gained from a recent test of an isolated plant, in the discharge of his routine duties as an engineer; but he hopes that the facts presented in this paper, may induce others of the Society better qualified than himself, to analyze the large losses of power and reconcile some of the discrepancies which were developed by the test referred to.

The Union Central Passenger Depot, at Cincinnati, is lighted under a contract with local parties, by an isolated Edison Incandescent Electric Light plant of a nominal capacity of eight hundred 16-candle

power lamps. The plant is owned and operated by the local company, which furnishes the necessary attendants, oil and waste for the motive machinery, and makes the lamp renewals; while the Depot Company owns and operates the boilers, and furnishes the steam to drive the engines. Therefore to the cost of the light service, as rendered in monthly bills by the owners of the electric light plant, must be added the gross cost of the steam consumed by the engines which operate the dynamos, to arrive at the actual cost to the Depot Company of the light.

The general terms of the contract under which the light is furnished, require the owners of the electric plant to develop an illumination the equivalent of gas, at the cost of gas; and further provides, that whenever desired, the Depot Company can make a test for comparative effects and costs of gas and electric light, for a period of sixteen days, using each illuminant for eight consecutive days of twenty-four hours each.

The steady and persistent introduction of arc and incandescent electric lighting, in places where under ordinary conditions the consumption of gas would be large, as railroad stations, hotels and public buildings, is viewed—to say the least—not with any feeling of pleasure by the gas companies of the large cities, and as a rule the exact information upon the actual cost, and the cost relative to gas, of incandescent electric lighting, is not as well known to the public or to gas companies, as it should be at the present time.

Constructors and operators of electric plants no doubt are possessed of full information upon the efficiency and cost of this mode of illumination, and seem disposed to retain this information as a profound secret in which laymen and the public have no interest.

During the month of October, 1887, the writer, by mutual agreement of the Union Central Passenger Depot Company, and the owners of the Edison Incandescent Electric Lighting plant in the station, made an eight-days test of the plant, in which he was assisted by Professor Thomas French, Jr., of the Cincinnati University, and Mr. Charles E. Jones, electrician, of Cincinnati. The former gentleman making the photometric tests of specimen lamps, taken from the fixtures in the building; and the latter, furnishing the galvanometer, and reading the currents required to maintain the specimen lamps under different conditions of pressure. Mr. Jones also made the necessary measurements to determine the resistance of the specimen lamps cold.

The motive power is obtained from two Buckeye high speed automatic non-condensing engines, which drive the dynamos through belts and pulleys on a jack-shaft, and are each of 50 nominal horse-power, and operating at a speed of 230 to 235 revolutions per minute.

In the following table are given the principal dimensions of the engines and the pulleys:

	East Engine.	West Engine.
Diameter of steam cylinders	10.02 inches.	10.03 inches.
Stroke of steam pistons	14.00 "	14.00 "
Diameter of piston rod	1.54 "	1.52 "
" pulley fly wheel	4.529 feet.	4.555 feet.
" driving pulley on jackshaft	2.497 "	2.516 "
" driving pulley on "	3.332 "	3.365 "
" dynamo armature pulley	1.1605 "	1.187 "
Ratio speed of dynamo to speed of engine,	5.207	5.132

The current generators consist of two Edison dynamos, each of 300 amperes capacity; each being furnished with a commercial ammeter, to indicate the current (amperes), and a Howell indicator to note the pressure of the current in volts.

The boilers furnishing the steam, three in number, are of the return tubular variety, and each of the following dimensions:

Diameter of shells	42 inches.
Length of shells	12 feet.
Number of tubes	40
Diameter of tubes (outside)	3 inches.

Grates, continuous under the three boilers, 43 inches long x 12 feet 7 inches wide.

Gas-house coke is used as fuel for the boilers.

The Edison lamps in continuous service, are all of 16 or 32 nominal candle-power; and are distributed through the depot building, train-shed, omnibus court, and storage rooms and stables of the American Express Company.

The test embraced: (*a*) a careful measurement of the performance and cost of operating the boilers; (*b*) the power developed by the driving engines, and the proportion of that power transmitted to the dynamos; (*c*) the power (electro motive force) of the currents generated by the dynamos; (*d*) the lamp service, taken by eight skilled observers, who made hourly rounds of the portion of the building in their respective charge, and noted the time, location and nominal candle-power, of each lamp in service. (In the train-shed, the lamps were operated in rows, as trains arrived and departed; and these were noted by the number

of the row and intervals in service); (*e*) the photometric test of forty-three specimen lamps, taken from the fixtures in and about the depot building and train-shed; (*f*) the test of specimen lamps, simultaneously for brilliancy and current, to estimate the proportion of electrical energy of the dynamos actually realized in the work of the lamps; and (*g*) a weighing of the positive plates of the voltameters; from the transfer of zinc in which the owners of the electric light plant render the monthly bills for lighting the depot and surrounding premises.

For convenience, the results of the tests are presented under the folling heads:

First.—Comparative cost of electric light and gaslight for equal effects.

Second.—Illuminating power of electric lamps, with changes in the angle of the filament or loop, and potential of current.

Third.—Performance of the voltameters and relation of their record to the light actually furnished.

Fourth.—Relation of the work of the engines to the work of the dynamos, and relation of the work of the dynamos to the work of the lamps.

Fifth.—Notes of explanation of the contents of the tables of observed and calculated data.

1. Comparative Cost of Electric and Gas Light.

The feed water was measured to the boilers, in two tight casks, containing at 65 degrees Fahrenheit 880.5 and 869.75 pounds respectively. These were charged alternately from the city mains, and the contents delivered through the feed-pump, to the boilers. The water record consisted of the number of casks consumed, the time required to exhaust each cask, and the temperature of the water, which also was the temperature of the feed to the boilers.

The coke, weighing 40 pounds to the standard bushel, was measured in uniform charges of 150 pounds, and two charges of 300 pounds were delivered to the firemen at each round; and no other or additional coke was permitted on the boiler-room floor, until the previous charges were thrown into the furnaces. The quality of the steam was taken with a very sensitive small Fairbanks' platform scale, graduated to weigh to two ounces, and a James Green United States Signal Service thermometer.

During the test of one hundred and ninety-two hours, or eight consecutive days (6 A.M., October 11th, to 6 A.M., October 19th), there were charged to the boilers, the following quantities:

	Pounds.
Coke to start fires..............................	782.75
Pine kindling, taken at one-half its weight in coke....................................	244.62
Coke to operate the boilers eight days...........	36 376.00

There are two batteries of boilers in the station, which are alternated in service every two weeks; whence the average daily consumption of fuel to start fires is 73.384 pounds; and the average daily consumption of fuel, including coke to operate the boilers, 4 620.384 pounds; and a consumption for the month of October, of 3 580.8 bushels, costing at seven cents per bushel, $250.66.

The consumption of water by the boilers from tank measurements for eight consecutive days, was 32 826.83 gallons; to which should be added, five per cent. for wetting the ashes, cleaning boilers and washing the boiler-room floor (taken from separate tap), or 1 641.34 gallons, making total consumption for eight days, 34 468.17 gallons, or a consumption for the month, of 133 564.15 gallons, costing at nine cents per 1 000 gallons, $12.02.

From the bills rendered the Depot Company by the owners of the electric light plant, for 1886, it seems the cost of light for the month of October, is 0.08615 of the cost for the whole year, and as the steam and consequently the cost for fuel and water will be proportional to the illumination furnished, then the cost of these items per annum will be as follows:

$$\text{Coke}\ldots\frac{250.66}{0.08615} = \$2\,909.53. \quad \text{Water}\ldots\frac{12.02}{0.08615} = \$139.53.$$

Add to this the wages of two firemen, at $45 per month each, or per annum $1 080.

Interest on boiler plant and fixtures, valued at $3 000, at five per cent., $150.

Depreciation of boiler plant, taken at 10 per cent. of valuation, or $300.

When the total cost of the steam per annum becomes $4 579.11.

From the tabulated statement of the lamp service during the eight days of test, the average illumination for the whole time is shown to have been equivalent to 185.15 lights of 16 actual candle-power each;

for which the charge for the interval of time according to the method of rendering bills for incandescent electric lighting would have been,

$$\frac{185.15 \times 16 \times 192 \times 45}{1000} = \$255.95.$$

and the annual charge,

$$\frac{255.95 \times 31}{8 \times 0.08615} = \$11\,512.55.$$

While it is the intention of the owners of the electric plant to make a charge of 45 cents per 1 000 standard candles of illumination, the bills are not based on the amount of light furnished, but on the deposit or transfer of zinc in the zinc voltameters, the assumption being that the zinc deposit, and consequently the average current passing through the voltameters bears a constant proportion to the total current in the circuit in which the voltameter is placed.

If it were convenient to state the actual amount of light furnished for any given length of time, as a certain hourly average number of 16 or 32 actual candle-power lights, it would be preferable to the present mode of charging for the electric light, but no meter has yet been devised—so far as the writer is aware—which will render an account of the lamps in service, and the average brilliancy (candle-power) of the lamps. If such an apparatus were invented, and could be depended upon to render an account of the lamp service, then its statement for the month (or any other interval of time), would be reduced to an average hourly service in thousand standard candles, and multiplied by the rate per thousand candles, upon which the contract for lighting was based. In this instance the rate is 45 cents per thousand candle-power of effect.

From the tabulated statement of the zinc voltameters the transfer of metal from the positive to the negative plates, for the one hundred and ninety-two hours of trial, in the aggregate, was: 21.829 grams, which, according to the formula of the owners of the electric lighting plant, is multiplied by the constant 17, to obtain the corresponding light furnished in thousand standard candles. (The standard candle, to be understood as burning at the normal rate of two (2) grains per minute, or 120 grains per hour.) From which the cost of the illumination (exclusive of steam) to the Depot Company, becomes: $C = 21.829 \times 17 \times .45 =$ \$167 for eight days, or \$647.09 for the month (October), corresponding to an annual charge of \$7 511.20, or a total cost for illumination per annum as follows:

Electric Light Company, charge	\$7 511.20
Cost of furnishing steam to operate the driving engine..................................	4 579.11
	\$12 090.31

The present price per 1 000 cubic feet for coal gas in Cincinnati is \$1.15 and by photometric test, according to the determinations of Professor French, has an illuminating power with a consumption of five (5) cubic feet per hour, of 16.94 standard candles, from which a light of 16 candle-powers would represent a consumption of 4.722 cubic feet per hour, and the cost of an illumination the equivalent of that furnished by the electric light for October would be,

$$C = 1.15 \times \frac{185.15 \times 4.722 \times 744}{1\,000} = \$748.03,$$

and an annual cost, assuming as before, that the light for the month of October represents 8.615 per cent. of the light for the whole year of, \$8 682.91 or a net annual saving over the electric light of \$3 407.40 corresponding to an increased cost for electric over gas lighting of 39.24 per cent.

If the electric light was charged for at the rate of 45 cents per thousand standard candles of *actual effect*, as the contract for the lighting really allows, then the cost per annum, including the outlay of the Depot Company for steam will be \$16 091.66, or \$7 408.75 over the cost for coal gas, corresponding to an increased cost of electric over gas lighting for same illuminating effect of 85.33 per cent.

Perhaps it will be more convenient to compare the costs of electric and gas-lighting in the following manner: To the rate of 45 cents per 1 000 candle-power of light furnished by the owners of the lighting plant add the cost (ascertained by trial) of the steam per 1 000 candle-power of light; or,

$$\text{Steam} \ldots\ldots\ldots\ldots \frac{101.54 \times 1000}{192 \times 185.15 \times 16} \times 100 = 17.85 \text{ cents.}$$

Cincinnati gas is shown to have an illuminating power of 16 candles for an hourly consumption of 4.722 cubic feet, and is sold at \$1 15 per thousand cubic feet, hence the cost of an illumination the equivalent of 1 000 standard candles burning at the normal rate,

$$\frac{4.722 \times 1.15}{16} \times 100 = 34 \text{ cents.}$$

Bringing the cost together we have:

Per 1 000 candle-power, Edison electric light.....63 cents.
Per 1 000 candle-power, Cincinnati gaslight......34 cents.

The average light service, and the average illuminating power of the electric lamps, was a matter of ocular demonstration, while the measurement of the illumination furnished by means of the zinc voltameters, depends upon the shunting of a small decimal (0.001) of the current in any circuit, through the meter connected in that circuit.

2. Illuminating Power of Electric Lamps, with Changes in the Angle of the Filament and Potential of Current.

Subsequent to the test for cost of electric light, a special investigation was made of three 16, three 32, and one 100 nominal candle-power lamps, selected from the specimen lamps tested by Professor French for brilliancy during the economy trial of the plant.

These lamps were tested simultaneously for strength of current and illuminating power; the filaments being placed successively in three positions at an angle of 45 degrees, at an angle of 90 degrees, and parallel with the plane of the photometer bar. The 100 candle-power lamp was tested in one position only, at an angle of 45 degrees to the plane of the bar.

The tests for brilliancy of the specimen lamps were made with a standard photometer from the American Meter Company, Philadelphia, the bar being graduated to give the candles power of the sample lights when compared with a single candle, burning at the normal rate of two grains of spermaceti wax per minute; and in order to diminish the ratio of electric to candle light, and keep the observation box over that portion of the scale, where slight differences of illuminating effect could be easily and safely read, two standard candles were burned, while testing the 16 and 32 candle-power lamps, and three candles while testing the 100 candle-power lamp. The standard light was furnished from imported candles, and the initial and final weights of the candles, and time to seconds, during which they were burned, carefully observed, to arrive at the exact rate of consumption of the spermaceti wax. The normal rate of consumption, as is well known, should be 120 grains per hour or two grains per minute, and the photometric observations were corrected to agree with the observed rate at which the sperm was burned for each test.

The galvanometer used in this test was a very perfect instrument, and was connected in the circuit callibrated for location and read by a very skillful observer.

The measured candle power of the specimen lamps in the following table are the means of the three positions in which the filaments were placed, excepting the 100 candle-power lamp as previously noted.

The potential difference given in the table was taken from the Howell indicator in use with the plant at the station.

COMPARISON OF LAMPS FOR CURRENT.

	DESIGNATION OF LAMPS.						NO MARK.
	B	D	C	F	A	E	
Nominal c. p.	32	32	32	16	16	16	100
Actual c. p.	62.44	30.90	30.16	36.33	21.10	16.36	144.75
Potential, Volts	99	99	99	99	99	99	99
Strength of current, amperes	1.25	1.189	1.310	0.773	0.816	0.694	2.986
Electrical energy in Watts	123.75	117.71	129.69	76.53	80.78	68.71	295.614
Watts, per nominal c. p.	3.867	3.678	4.053	4.783	5.049	4.294	2.956
Watts, per actual c. p.	1.982	3.809	4.300	2.106	3.828	4.199	2.042
Resistancé, cold, Ohms	143.5	156.0	135.0	239.0	224.0	241.0	Not taken.
Resistance, hot, Ohms	79.2	83.3	75.57	128.0	121.3	142.6	33.15

The 16 and 32 c. p. lamps were tested October 22, and the 100 c. p. lamp October 21, 1887.

AVERAGES.

Nominal c. p.	Actual c. p.	Current amperes.	WATTS.		RESISTANCE.	
			Per nominal c. p.	Per actual c. p.	Cold.	Hot.
16	24.597	0.7610	4.709	3.377	234.7	130.63
32	41.166	1.2497	3.866	3.363	144.8	79.36
100	144.75	2.986	2.956	2.042	....	33.15

The resistance in ohms of the lamps hot is expressed as follows:

Let $A =$ strength of current in amperes,
$P =$ the potential difference in volts, and,
$O =$ the resistance in ohms,

then $O = \dfrac{P}{A}$

The Watts or units of electrical energy required to maintain the brilliancy of the lamps is obtained by multiplying the current by the potential, or $W = A \times P$, where $W =$ Watts.

In the table of averages it will be noted that the Watts per actual candle-power of lamps is practically the same for 16 and 32 candle-power lamps, while for the 100 candle-power lamp, the electrical energy per actual candle-power is less than 61 per cent. of the energy required by the smaller or weaker lamps. The relative energy required by the 100 candle-power lamp, agrees with the general assertion of electric light operators that a given illumination costs less with lamps of high than low candle-power. But when we turn to the electrical energy

required to maintain the 16 and 32 candle-power lamps, it seems that in this respect there is nothing in favor of the lamps of higher candle-power, and that it costs just twice as much for one 32 candle-power light as for one 16 candle-power light. Of course the cost of lamp renewals is relatively less, when the given illumination is produced with one-half the number of 32 candle-power lights, because the latter, while lasting no longer in service, costs less than twice as much as lamps of 16 nominal candle-power. The relative cost being about as 50 to 65.

The (*D*) 32 candle-power lamp was tested for current and brilliancy, for different conditions of potential, with the following results:

Potential.	Amperes.	Candle-power.
99—	1.050	23.94
99	1.189	31.20
99+	1.268	43.89

No test was made of the Howell indicator from which the potential was read, and the exact value of the graduations, above and below the normal position of the index (99 volts), is unknown. But the whole range of the scale was supposed to be from 93 to 105 volts, from which it appears that a slight change in the potential—such a change as frequently takes place in the operation of the plant, even with reasonably constant manipulation of the resistance box—there is a very great change in the illuminating power and cost of furnishing the light.

The reduction of the resistance reduces the potential and strength of current, and diminishes the power required to drive the dynamos, and prolongs the life of the lamps; but with a loss in the illuminating effect. So long as the illumination is paid for upon the weight of zinc transferred in the voltameters, there may be no serious objection to the variable resistance and potential, and variation in the actual candle-power of the lamps. But should the light be charged for upon the basis of a given average number of lamps, of a given average candle-power, for a given number of hours, then it might operate against the interests of the consumer, in that he may be paying for considerably more light than he receives.

An observer of a lamp giving 43 candle-power actual effect, could distinguish the reduction, if its power was diminished to 24 candles, but could form no correct conception of the measure of reduction, while the life of the lamp would be greatly prolonged by the change of power, and the cost of producing it diminished from 20 to 25 per cent.

Referring to the table, a comparison of the currents required to maintain the lamps, with constant potential (99 volts), shows that the electrical energy, and consequently the cost, is about the same for weak and strong lights; or while the illuminating power of the lamps steadily diminishes with age (use), the strength of current to maintain its brilliancy is substantially constant. Indeed, it may be shown by the data

of this test that the electrical energy and cost of the light becomes greater with the diminished candle-power, thus:

Nominal candle-power lamps.	Actual candle-power lamps.	Potential volts.	Current amperes.	Electrical energy. Watts per actual candle-power.
32 (*B*)	62.44	99	1.25	1.981
32 (*D*)	30.16	99	1.31	4.300
16 (*F*)	36.33	99	0.773	2.106
16 (*E*)	16.36	99	0.694	4.199

Upon the whole the average current and electrical energy for the tested lamps is substantially the same as for the new lamps of each candle power, thus:

32 candle-power,	lamp (*B*) new lamp......	1.25 amperes,	123.75 Watts.
32 "	lamps average all........	1.2497 "	123.72 "
16 "	lamp (*F*) new lamp......	0.773 "	76.53 "
16 "	lamps average all........	0.761 "	75.34 "

From which it appears that while the electrical energy and cost of maintaining the brilliancy of the lamp, during its life, is reasonably constant, the cost of the light furnished is in an increasing ratio; or, in other words, while the lamps are new the cost for a given illumination is but one-half of the cost for the same illumination after the lamps have been in service for a few weeks. With gas or kerosene a given constant consumption of the illuminating material furnishes a constant power of light, while a diminished brilliancy of light is always accompanied by a reduced consumption of material and a corresponding reduction of cost; with the electric light of the Edison incandescent system, while the power of the light steadily diminishes with use, the cost increases in a corresponding ratio.

Of thirty-three 16 nominal candle-power lamps tested by Professor French, the average brilliancy, when placed at angle of 45 degrees to the plane of the photometer bar, was........	16.99	candle-power.
When placed at angle of 90 degrees to the plane of the bar..............................	14.26	"
And when placed parallel with the bar..........	17.05	"
Of the ten 32 nominal candle-power lamps, the average brilliancy, when placed at an angle of 45 degrees to the plane of the photometer bar, was..................................	41.08	"
When placed at an angle of 90 degrees to the plane of the bar..............................	34.13	"
And when placed parallel with the bar..........	40.66	"

The angle of maximum illuminating power for the filament of an electric lamp is supposed to be 45 degrees to the plane of the photometer bar; but from these tests it seems there is very little difference in the

average effects of the lamps when the filament is placed parallel with or at an angle of 45 degrees to the bar. But with the filament placed flatwise, or at right angles to the bar, there is a loss of illuminating power from 17 to 18 per cent.

The extreme sensitiveness of the lights to slight changes in the potential, or strength of current, was clearly shown during the special test for brilliancy and current, the power of the light frequently changing from 20 to 30 per cent. while taking an observation. To the writer it seems that this range and rapidity of variation in the brilliancy of the light is a dangerous feature of the incandescent lighting system, especially if the light should be applied to any close work as reading, writing, drawing, engraving and similar occupations. For general illumination the fluctuations of candle-power may not form a serious objection; but the hurtful influence to the eye of a light which is rapidly changing in illuminating power, is generally recognized by occulists.

The average illuminating power of the thirty-three 16 nominal candle-power lamps tested for brilliancy, was 16.199 actual candle-power; and for the ten 32 nominal candle-power lamps, 38.637 actual candle-power. From which the 16 nominal candle-power lamps in service during the test of eight days should be increased in the ratio of 1.0124, and the 32 candle-power lamps, 1.2074, for actual illumination furnished.

The general average hourly number of lamps in service, were:

Of 16 candle-power lamps........................	111.961
Of 32 candle-power lamps........................	29.733

and reducing the 32 nominal candle-power lamps to an equivalent number of 16 candle-power lamps, and increasing both in above ratios, we have as the actual average hourly light service, 185.15. 16 candle-power lights, costing, as previously stated, for an interval of 192 hours, $268.54.

3. Performance of the Voltameters.

The voltameters used by the owners of the electric lighting plant to measure the service to the Depot Company, contain two thin zinc plates attached to the positive and negative poles of the cup; the transfer of the zinc from the positive to the negative plate—for any given length of time—being taken as the difference of weight of the losing plate before and after the interval.

In the following table are given the sums of the losses of zinc between 6 A.M. October 11th, and the same hour October 19th, for forty voltameters, divided as follows:

5 six-light meters.
10 twenty-five-light meters.
21 fifty-light meters.
4 one hundred-light meters.

The light mentioned in the designation of the voltameters, means

nominal 16 candle-power lights, from which the meter capacity of the electric lighting plant at the Union Depot is equivalent to 1.730 lamps of 16 candle-power.

WEIGHT OF ZINC PLATES OF VOLTAMETERS.

	Grams.
SIX LIGHT METERS (5):	
Aggregate weight 5 plates, October 11th	106.770
" " " 19th	106.371
Difference	0.399
TWENTY-FIVE LIGHT METERS (10):	
Aggregate weight 10 plates, October 11th	761.005
" " " 19th	759.118
Difference	1.887
FIFTY LIGHT METERS (21):	
Aggregate weight 21 plates, October 11th	3 068.348
" " " 19th	3 055.427
Difference	12.921
ONE HUNDRED LIGHT METERS (4):	
Aggregate weight 4 plates, October 11th	1 252.760
" " " 19th	1 246.138
Difference	6.622
Total deposit for 192 hours	21.829

During the weighing of the zinc plates of the voltameters, the phenomena (which as the writer was informed is not infrequent) developed, of the positive plate gaining instead of losing in weight; showing an apparent reversal of current for meters Nos. 23, 24, 31, 32 and 39. The custom in cases of this kind, being to take the gain of the plate in weight, as the measure of zinc transferred.

Reducing the total deposit of zinc, for eight days, to the average transfer per second, we have, $\frac{21.829}{192 \times 60 \times 60} = 0.00003158$ gram; corresponding according to accepted authority in such matters (*), to an average current of $\frac{0.00003158}{0.0003392} = 0.0931$ ampere.

The proportion of current through the shunts of the voltameters, is $\frac{1}{1000}$ of the entire current; hence total average current for the time of trial, was....93.1 amperes.

* Practical Electricity, by W. E. Aryton, F.R.S., pages 11-12.

The average illumination produced, was equivalent to 185.15 lights of sixteen candle-power, and from the record of the voltameters the current per 16 candle-power of actual light, was

$$C = \frac{93.1}{185.15} = 0.50283 \text{ ampere.}$$

It is claimed that the current for a nominal 16 candle-power lamp, is about 0.75 ampere; and that this is correct for average lamps, when the potential difference is maintained at 99 volts, is shown earlier in this paper, where the average for three specimen 16 candle-power lamps of different ages, is 0.761 ampere. But the average illumination of 185.15 16 candle-power lamps, from the observation of the lighting service during the test of eight days, are not average lamps but actual lights of 16 candle-power each; and by reference to the tabular statement upon page 150, it will be seen that while the average current for 16 candle-power lamps is 0.761 ampere, the average illuminating power of the tested lamps was 24.597 candles; hence by proportion the current for a light of 16 actual candle-power, should be, $\frac{16 \times 0.761}{24.597} = 0.495$ ampere; or substantially the same current as found from the transfer of zinc in the voltameters.

Adopting the formula employed by the owners of the Electric Lighting plant, the transfer of zinc in the voltameters was held to represent an illumination during the test of 192 hours, of $21.829 \times 17 \times 1000 =$ 371.093 candle-power, or 1 932.77 standard candles per hour; which reduced to lights of 16 candle-power, will be 120.736.

The average lamp service, from actual observation, has been stated as 185.15 lights of 16 actual candle-power; from which it seems that the voltameters accounted for only $\frac{120.736 \times 100}{185.15} = 65.21$ per cent. of the actual illumination. But the strength of current per actual candle-power of light furnished, according to the voltameters, was 0.0314 ampere, and the strength of current per actual candle-power of light, from the special test of lamps for brilliancy and current was, for 16 candle-power lights $\frac{3.37}{99} = 0.0341$ ampere; and for the 32 candle-power lights $\frac{3.363}{99} = 0.0339$ ampere; or a mean of 0.034 ampere for both; hence the voltameters—upon the assumption that $\frac{1}{10}$ per cent. of the total current was shunted through them, and that the accepted rate of transfer of zinc per second per ampere of current (0.0003392 grams) is correct—really accounted for $\frac{0.0314}{0.034} \times 100 = 92.35$ per cent. of the total work of the lamps.

Considering the fact of the test for electrical energy required to maintain the lamps at brilliancy, being made subsequent to the general test

for economy of lamp service and illuminating power of lamps; and that the test (for electrical energy) embraced a small number of specimen lamps, the agreement of the work of the voltameters with the observed service of the plant, is reasonably close.

It seems to the writer, if the rate of 45 cents per thousand standard candles of illumination, as charged by the owners of the Electric Lighting plant, is *bona fide*, that the constant 17, by which the deposit or transfer of zinc in the voltameters is multiplied, is in error; and should be increased in the ratio of $\frac{1}{0.6521} = 1.5335$, or should be 26.07; adopting which as the proper constant, the illumination accounted for by the voltameters, during the test, was the equivalent of $21.829 \times 26.07 \times 1\,000 = 569\,082.03$ standard candles; corresponding to an average hourly service of 185.28 lights of 16 candle-power.

From the work of the voltameters, and the special test of specimen lamps for brilliancy and current, the fact seems to be established, that for average lamps—the electrical energy per unit of light is substantially alike for 16 and 32 candle-power lamps; and the electrical energy per unit of light (candle-power), is practically constant for any given lamp operating under different conditions of potential; that is to say, with a 16 nominal candle-power lamp, yielding with one potential an illumination of 16 candles, the electrical energy is increased or diminished in direct proportion, as the potential and power of the light is increased or diminished. This statement, however, must not be confounded with a previous statement, that as the age (use) of the lamp increases the electrical energy per actual candle-power of the light also increases, and that the total energy for any one lamp does not vary materially between its highest and lowest power of illumination, but must be understood as applying to a given lamp at a given age.

4. Relation of the Work of the Engines to the Work of the Dynamos, and Relation of the Work of the Dynamos to the Electrical Energy realized in the Lamps.

During the five days of the test (October 13th–17th inclusive), while the engines were being indicated for power, the average indicated horse-power—for the day service—was 20.306, and the average illumination reduced to lights of 16 actual candle-power each, was 95.52.

No tests were made for friction of the engines and jack-shafts, but from tests of similar engines and connectors, it is assumed that of the indicated power during the day service the friction losses were as follows:

Friction of engine, without load.......	1.09	horse-power.
" " additional due load.	0.96	"
" " jack shaft..........	0.61	"
Total friction..................	2.66	"

or about 13 per cent. of the indicated power was absorbed by frictional losses, leaving 17.645 horse-power to be transmitted to the dynamos; of this, however, 3.516 per cent. was lost by slip of belts, and the net power actually delivered to the dynamos was approximately 17.02, or expressed in terms of electrical energy 12 696.92 Watts.

The lamp service during the daylight hours is given above as the equivalent of 95.52 lights of 16 candle-power; and the electrical energy per lamp, $3.377 \times 16 = 54.032$ Watts; and the electrical energy represented by the actual illumination was:

$$\text{Watts} = 95.52 \times 54.032 = 5\,161.136,$$

or of the net work of the driving engines transmitted to the armature of the dynamos, only 40.64 per cent. was realized in the work of the lamps.

No statement could be made of the electrical energy developed by the dynamos during the day service of the plant; from the fact of the ammeters being so constructed that considerable current was produced before the index would move from the zero point, and this additive quantity being unknown, readings of the ammeters below 100 amperes could not be made with any degree of accuracy, and were accordingly omitted in making the record of trial.

The statement of the independent work of the dynamos is therefore of necessity limited to the night service of the plant, when the average reading of both ammeters was over 160 amperes, and the least reading of either, 110 amperes; the indications of the instruments, being accepted as reasonably reliable after the index passed the first graduation (100 amperes), when the unknown strength of current required to start the needle from zero was eliminated.

In the following table are given the average indications of the ammeters, during the night service, from October 13th to 17th, inclusive; corresponding with the intervals when the driving engines were being indicated for power.

STRENGTH OF CURRENT.

Date.	East Dynamo Amperes.	West Dynamo Amperes.	Total Amperes.
October 13th	163.330	150.000	313.330
" 14th	175.909	160.000	335.909
" 15th	166.875	181.000	347.875
" 16th	150.000	147.500	297.500
" 17th	159.000	159.000	318.000
Average			322.523

The average indicated horse-power during the night service was 70.705, of which the losses by friction are estimated as follows, for each engine:

Friction of engine, without load........	1.0902	horse-power.
" " additional due load.	1.7131	"
" " jack-shaft	0.6092	"
Total friction, one engine.........	3.4125	"
Total friction, both engines.......	6.8250	"

in round numbers, 9.65 per cent. of the indicated power, leaving 63.88 horse-power to be transmitted to the dynamos—less the slip of belts—3.516 per cent., or 2.25 horse-power; and net work of engines actually delivered to the dynamos, 61.63 horse-power; or, in terms of electrical energy, 45 975.98 Watts.

The work of the dynamos during the night service was carefully observed, with the following results:

Average difference of potential.........	99 volts.
Average strength of current............	322.523 amperes.

and average electrical energy....... $322.523 \times 99 = 31\,929.777$ Watts, plus losses not measured between the dynamos and testing instruments. (The ammeters and volt indicators were closely connected with the dynamos, with no intermediate circuit connections, and the losses must have been trifling).

Of the net power of the engines actually transmitted to the dynamos; $\frac{31\,929.777}{45\,975.98} \times 100 = 69.45$ per cent., was realized in the work of the dynamos. This difference of energy, or loss of useful work of over 30 per cent., may be separated into two quantities; that due to friction of the dynamo armatures in their bearings, and that due to the resistance in the field of the dynamos. Allowing 7 per cent. of the reduction of work for the friction of the armatures, then 23 per cent. is absorbed in overcoming the resistance within the dynamo.

The lamps in service during the intervals, when the work of the engines and the work of the dynamos were simultaneously measured, were equivalent to 429.319 lights of 16 candle-power; and the electrical energy represented by the actual illumination was:

$$429.319 \times 54.032 = 23\,196.96 \text{ Watts;}$$

or of the net work developed by the dynamos:

$$E = \frac{23\,196.96}{31\,929.777} \times 100 = 72.65 \text{ per cent.,}$$

was accounted for in the electrical energy of the lights.

Omitting the mechanical losses due to friction of engines, extra friction due to load and friction of jack-shafts, only a trifle over one-half (50.45 per cent.) of the net power transmitted to the dynamos, is actually realized in the work of the lamps; and taking the day and night service together, which, for commercial purposes, is the proper mode of estimating the net efficiency of a plant for artificial illumination; then of the net engine power delivered to the dynamos, but 45.5 per cent. is actually realized in the illumination furnished.

This test was made to ascertain, not particular losses, or causes of loss, but the relative cost of electric and gas lightng, and no other data were taken than enough to determine this question, excepting the special test of specimen lamps (before noted) for brilliancy and current.

The writer is not sufficiently familiar with current literature upon the incandescent electric light to know how favorably the efficiency of this plant as a whole, compares with other plants which have been tested; but he has been assured by others interested in lighting, that this Central Depot plant was of a representative character, and that upon test it would compare favorably with any other isolated Edison plant in similar service. If this is true, then the large losses between the driving engines and the current generators, and between the motive power and the illumination produced, should be the means of stimulating electrical engineers to a remedy; the effect of which will of course be to reduce the cost of producing the light.

Constructors of Edison electric lighting plants are accustomed to estimate eight 16 actual candle-power lights to each indicated horse-power of driving engines; which, if realized in practice, would have a marked effect on the cost and relative economy of incandescent electric lighting; but from this test, the ratio of lamps of 16 nominal candle-power, to each indicated horse-power of engine was

For the day service, including October 16th (Sunday)	4.357
For the day service, omitting October 16th.........	4.728
For the night service, including October 16th.......	5.547
For the entire daily service (24 hours), including October 16th....................................	4.952
For the entire daily service (24 hours), omitting October 16th....................................	5.136

It will probably be more accurate to state the number of lamps or lights of 16 actual candle-power per indicated horse-power of driving engines. The average indicated horse-power of engine for the day service, for five days, including October 16th, was 20.306, and the average actual illumination for same time, 95.52 lights of 16 candle-power; and lamps per indicated horse-power:

$$L=\frac{95.52}{20.306}=4.704.$$

The average indicated horse-power for the night service, including October 16th, was 70.707, and average actual illumination for same time, 429.319 lights of 16 candle-power; and lamps per indicated horse-power:

$$L=\frac{429.319}{70.707}=6.072,$$

or, for continuous day and night service, 5.388 lamps of 16 candle-power each, per indicated horse-power of engine. From which it will be seen

that, even under the most favorable conditions of load and service for this plant, the number of lamps per horse-power is twenty-five per cent. below the proportion of lamps to power assigned by constructors.

Incidental to the performance of the boilers which furnished the steam for the engines, the following data are given.

Non-combustible in the coke..............	5.21 per cent.
Steam per pound of coke, from and at 212 degrees Fahr.	8.38 pounds.
Steam per pound of coke, from temperature of feed water (65 degrees Fahr.).......	7.266 "

The efficiency of the steam from thirty-four tests was 0.97564, equivalent to an entrainment of 3.336 per cent. of the total feed water supplied to the boilers.

5. Notes of Explanation of the Tables of Data.

The record of the lights in hourly service in and about the Depot building, may for convenience, be divided into (*a*) lights throughout the building, not including the train shed, omnibus court, Smith street cross lights, or American Express building; (*b*) lights in the train shed, omnibus court, cross lights over the railroad tracks west of the station; (*c*) lights in the American Express building.

Lights (*a*) and (*b*) were taken by two sets of observers, day and night, during the eight days of trial. Lights (*c*) were taken for the nights of October 18th, 19th and 20th, and averaged for the trial. Lights (*a*) were read hourly, and averaged for the day service, 6 A.M. to 5 P.M., and for the night service, 6 P.M. to 5 A.M., and for shorter day and night intervals, to correspond with the intervals during which the work of the engines and current generators were being measured. Lights (*b*) were read by rows from the switch-board in the train shed, for the intervals and total time of service for each night of the test. Lights (*c*) were read for intervals and total time of service each night, and also by hours, from 6 P.M. to 6 A.M.

Tables "A" to "E," inclusive, embrace the lamps in service during the entire trial, in detail as follows: Table "A" contains the record of the day service of lamps in the Depot building, from 6 A.M. to 5 P.M. for each of the eight days of test. Table "B" contains the record of the night service of lamps in the Depot building, 6 P.M. to 5 A.M., for each day of the eight days of test.

Referring to Tables "A" and "B," columns 1 to 14, inclusive, are explained by the headings; column 15, Table "A," contains the average hourly lamps for that portion of the day, 10 A.M. to 5 P.M., inclusive, during which the power of the driving engines was measured; and column 16 contains the average of lamps per hour for the entire day (12 hours). Column 15, Table "B," contains the average of hourly lamps for that

portion of the night, 6 P.M. to 10 P.M., inclusive, during which the power of the driving engines and work of the dynamos were measured, and column 16 contains the average lamps per hour for the entire night (12 hours).

Table "C" contains the lamps in service in the warerooms and stables of the American Express Company, for the dates noted; divided into sub-tables, "C" 1, containing the hourly record of lamps in service; "C" 2, containing the average hourly service, between 5 P.M. and 7 A.M.; and "C" 3, the average hourly service between 5 P.M. and 10 P.M., corresponding with the hours during which the work of engines and dynamos were measured.

Tables "D" and "E" exhibit the lamp service for the eight days of test in the train shed. (This embraces the rows of lights over the railroad tracks, the lights in the lobby at the east end of the shed known as the gate row, the lights in the omnibus court at the easterly entrance of the depot, and the lights along the tracks west of the depot. These lamps are divided into twelve rows or series, and controlled from a single switch-board in the train shed). In recording the service the observer noted the rows and times of opening and closing the circuits on the switch-board; and the tables contain the total time for which each particular series of lamps were in use for each day of the test.

The form of subdivisions of tables "D" and "E" is alike for all. At the head is given the date for which such table is compiled. Column 1 contains the number on the switch-board of the series or row of lamps; column 2 contains the number of lamps in the row; column 3 contains the hours and minutes of service; column 4 contains the lamp hours, and is the product of the lamps in the series multiplied by the hours and decimal of service; column 5 contains the average lamps in service per hour, estimated for the hours between which the respective series of lamps are first lighted and last extinguished. Series 1, 2, 3, 4, 5, 6, 8, 9, 11 and 12 are taken for six hours, series 7 for twelve hours, and series 10 for seven hours.

Table "F" exhibits the total average lamps for the night service, from 6 P.M. to 6 A.M., and brings together the lamps for the depot building, American Express Company building (which is furnished with light from the electric lighting plant), and train shed. The headings of the columns sufficiently explain their contents.

Table "G" exhibits the same data as Table "F," but for the interval of time embraced between the hours of 6 P.M. and 10 P.M.

Table "H" contains the average revolutions of the driving engines, arranged for the day and night service of the plant.

Table "J" exhibits a comparison of the speeds of engines and dynamos for same intervals of time, and shows the slip of belts, or loss of motion between the engine and dynamo pulleys. The contents of the columns will be understood from the respective headings. (Assuming

no slip of belts, the revolution of the dynamo armature should be directly as the diameters of driving and inversely as the diameters of driven pulleys.) The pulleys were carefully measured for circumference with a Chesterman tape, and found to be of diameters as stated earlier in this paper. The speeds given in column 5 of this table are obtained by multiplying the known speeds of engines by 5.207 or 5.132 respectively, and the percentage of engine speed realized by dynamo, column 6, is had by multiplying the values in column 4 by 100, and dividing by the values giving in column 5.

Table "K" exhibits the indicated horse-power of the driving engines, arranged for the day and night service of the plant.

Table "L" exhibits the performance of the boilers during certain intervals of the eight days of the trial, divided into two sub-tables, of observed and calculated data.

Table "M" exhibits the calorimeter work for test of the quality of steam furnished by the boilers, and contains the averages of thirty-four observations of weights of steam condensed, water heated by the steam, and initial and final temperatures of water heated; the co-efficient of saturated steam 0.9666 was used to reduce the apparent to the actual evaporation in Table "L."

Table "N" exhibits in parallel columns the indicated horse-power of the engines; average lamps in hourly service for the intervals during which the power of engines was measured; equivalent lamps of 16 nominal candle-power; and number of 16 nominal candle-power lamps per indicated horse-power of engines. The actual 16 and 32 candle-power lamps given in column 4 are taken from the hourly records shown in detail in tables "A" to "E;" and the 32 candle-power lamps are reduced to the equivalent number of 16 candle-power lamps in the ratio of two to one, in accordance with the results given under head of "Illuminating Power of Lamps," wherein it appears that the electrical energy per actual candle-power of the specimen 16 and 32 nominal candle-power lamps tested for brilliancy and current are alike, or 3.377 and 3.363 Watts respectively.

TABLE "A."

Hourly Record of Day Lamps.

Date.	Lamps. Nominal c. p.	Time. A.M. 6	7	8	9	10	11	12	P.M. 1	2	3	4	5	Average part day.	Average part night.
Oct. 11.	16	85	71	94	71	70	80	64	81	90	91	92	126	......	84.58
" "	32	6	5	8	10	7	9	8	9	12	10	12	10	......	8.83
" 12.	16	92	73	89	85	83	82	72	69	82	85	87	116	......	84.58
" "	32	6	5	8	10	10	10	8	9	10	10	10	10	......	8.83
" 13.	16	91	75	72	73	70	70	66	62	74	78	83	106	81.20	76.66
" "	32	7	5	7	9	9	10	7	8	9	9	10	11	9.73	8.42
" 14.	16	100	68	86	99	86	86	70	61	76	85	85	118	86.06	85.00
" "	32	7	5	11	10	10	10	7	8	8	9	10	10	9.29	8.75
" 15	16	90	79	83	76	83	79	64	67	80	84	87	111	85.51	81.92
" "	32	5	7	9	11	11	12	11	10	9	12	12	10	11.36	9.916
" 16.	16	87	66	61	39	29	28	28	30	29	29	31	34	29.86	40.92
" "	32	10	6	7	6	6	6	5	5	5	5	4	4	4.85	5.75
" 17.	16	113	76	86	92	78	77	66	63	72	81	87	113	79.85	83.666
" "	32	8	7	9	10	10	11	9	10	10	10	10	14	10.57	9.83
" 18.	16	97	72	80	82	75	82	74	71	84	88	93	142	......	86.666
" "	32	7	6	8	10	10	10	9	10	10	10	12	14	......	9.666

TABLE "B."

Hourly Record of Night Lamps.

(Exclusive of Train Shed and Express Office.)

Date.	Lamps. Nominal c. p.	Time. P.M. 6	7	8	9	10	11	12	A.M. 1	2	3	4	5	Average from 6 to 10.	Average, whole night.
Oct. 11	16	248	231	226	182	138	125	78	76	76	76	74	77	......	133.91
and 12	32	35	34	35	32	23	21	5	5	5	5	6	7	......	17.75
Oct. 12	16	276	256	255	247	214	162	82	78	79	77	78	81	......	157.08
and 13	32	39	38	35	35	29	23	6	5	5	5	5	8	......	19.42
Oct. 13	16	278	231	229	212	182	129	91	77	75	75	73	77	226.4	144.08
and 14	32	31	31	31	29	27	21	7	7	7	7	7	7	29.8	17.666
Oct. 14	16	252	225	230	209	181	135	84	81	77	77	78	80	219.4	142.41
and 15	32	32	31	31	29	22	19	8	7	6	6	6	8	29.0	17.08
Oct. 15	16	261	226	222	213	184	149	88	82	79	77	76	80	221.2	144.75
and 16	32	29	31	31	31	22	19	5	5	5	6	6	7	28.8	16.42
Oct. 16	16	210	220	216	199	176	120	82	81	77	77	77	80	204.2	134.58
and 17	32	30	29	29	27	27	19	7	7	7	7	7	9	28.4	17.08
Oct. 17	16	277	226	228	217	179	134	86	81	78	77	76	78	225.4	144.75
and 18	32	31	30	30	28	22	20	7	7	7	6	6	7	28.2	16.75
Oct. 18	16	266	229	157	218	187	142	88	82	79	78	78	81	......	140.4
and 19	32	32	31	31	29	25	20	5	5	5	5	5	7	......	18.0

TABLE "C."

American Express Office and Stable Lamps.

Record of Lamps by Hours.

Date.	Lamps. Nominal c. p.	P.M.							A.M.					
		6	7	8	9	10	11	12	1	2	3	4	5	6
Oct. 18	16	11	10	2	1	1	1	1	0	0	0	5	5	5
and 19	32	5	7	2	2	2	2	2	2	2	2	4	4	4
Oct. 19	16	10	10	5	1	1	1	0	0	0	0	4	5	2
and 20	32	6	7	2	2	2	2	2	2	2	2	3	3	3
Oct. 20	16	10	10	4	1	1	1	0	0	0	0	4	5	2
and 21	32	6	7	2	2	2	2	2	2	2	2	4	4	4

Between 5 p.m. and 7 a.m.

October 18–19.	Average hourly	16 c. p.	lamps	......3.215
	"	32 c. p.	"	......3.291
October 19–20.	"	16 c. p.	"	......3.194
	"	32 c. p.	"	......2.263
October 20–21.	"	16 c. p.	"	......3.125
	"	32 c. p.	"	......3.402
Average for three nights,		16 c. p.	"	......3.178
" "		32 c. p.	"	......2.985

Between 5 p.m. and 10 p.m.

October 18.	Average hourly	16 c. p.	lamps	..........4.73
	"	32 c. p.	"	..........3.50
October 19.	"	16 c. p.	"	..........5.46
	"	32 c. p.	"	..........4.05
October 20.	"	16 c. p.	"	..........5.28
	"	32 c. p.	"	..........3.70
Average for three nights,		16 c. p.	"	..........5.157
" "		32 c. p.	"	..........3.75

TABLE "D."

Record of Lamps. Train Shed.

All 32 c. p. Lamps.

October 11th.						October 12th.					
No. of Row.	Lamps.	Service. Hrs.	Service. Min.	Lamp Hours.	Average Lamps per Hour.	No. of Row.	Lamps.	Service. Hrs.	Service. Min.	Lamp Hours.	Average Lamps per Hour.
1	7	3	23	23.681	3.947	1	7	3	00	21. 00	3.500
2	7	1	33	10.850	1.808	2	7	2	47	19.481	3.247
3	7	4	08	28.931	4.822	3	7	4	04	28.462	4.744
4	6	1	54	11.400	1.900	4	6	2	35	15.498	2.583
5	7	3	04	21.462	3.577	5	7	3	10	22.162	3.694
6	6	0	57	5.700	0.950	6	6	2	55	17.496	2.916
7	6	11	37	69.696	5.808	7	8	12	14	97.864	8.155
8	6	1	38	9.798	1.633	8	6	3	17	19.698	3.283
9	9	2	23	21.447	3.574	9	9	3	09	28.350	4.725
10	7	5	37	39.312	5.616	10	7	6	17	43.981	6.283
11	6	2	40	15.996	2.666	11	2	6	02	12.066	2.011
12	14	4	04	56.924	9.487	12	14	4	03	56.700	9.450

October 13th.						October 14th.					
No. of Row.	Lamps.	Service. Hrs.	Service. Min.	Lamp Hours.	Average Lamps per Hour.	No. of Row.	Lamps.	Service. Hrs.	Service. Min.	Lamp Hours.	Average Lamps per Hour.
1	7	2	41	18.781	3.130	1	7	1	55	13.412	2.235
2	7	1	58	13.762	2.294	2	7	1	37	11.312	1.885
3	7	4	12	29.400	4.900	3	7	3	53	27.181	4.530
4	6	2	34	15.396	2.566	4	6	2	36	15.600	2.600
5	7	2	07	14.840	2.473	5	7	3	31	24.612	4.102
6	6	1	59	11.898	1.983	6	6	3	45	22.500	3.750
7	8	11	38	93.064	7.755	7	8	12	13	97.728	8.144
8	6	3	09	18.900	3.150	8	6	2	44	16.398	2.733
9	9	2	56	26.397	4.399	9	9	2	58	26.694	4.450
10	7	6	31	45.612	6.516	10	7	5	42	39.900	5.700
11	4	5	02	20.132	3.355	11	4	4	26	17.732	2.955
12	14	4	48	67.200	11.200	12	14	4	33	63.700	10.616

Note.—Row 7 averaged for 12 hours. Row 10 averaged for 7 hours. All others averaged for 6 hours.

TABLE "E."

Record of Lamps—Train Shed.

All 32 c. p. Lamps.

October 15th.						October 16th.					
No. of Row.	Lamps.	Service. Hrs.	Service. Min.	Lamp Hours.	Average Lamps per hour.	No. of Row.	Lamps.	Service. Hrs.	Service. Min.	Lamp Hours.	Average Lamps per hour.
1	7	2	44	19.131	3.188	1	7	1	54	13.300	2.216
2	7	1	16	8.862	1.477	2	7	1	02	7.231	1.205
3	7	2	29	17.381	2.897	3	7	3	10	22.162	3.694
4	6	1	51	11.100	1.850	4	6	1	40	9.996	1.666
5	7	3	06	21.700	3.616	5	7	2	36	18.200	3.033
6	6	3	44	22.398	3.733	6	6	2	11	13.098	2.183
7	8	11	45	94.000	7.833	7	8	11	13	89.728	7.477
8	6	2	49	16.896	3.816	8	6	2	04	12.396	2.066
9	9	2	38	23.697	3.949	9	9	2	37	23.544	3.924
10	7	6	17	43.981	6.283	10	7	5	38	39.431	5.633
11	4	5	01	20.064	3.344	11	4	5	29	21.932	3.655
12	14	4	05	57.162	9.527	12	14	4	14	59.262	9.877

October 17th.						October 18th.					
No. of Row.	Lamps.	Service. Hrs.	Service. Min.	Lamp Hours.	Average Lamps per Hour.	No. of Row.	Lamps.	Service. Hrs.	Service. Min	Lamp Hours.	Average Lamps per hour.
1	7	2	10	15.162	2.527	1	7	2	35	18.081	3.013
2	7	1	13	8.512	1.419	2	7	1	46	12.362	2.060
3	7	3	28	24.262	4.044	3	7	3	46	26.362	4.394
4	6	1	51	11.100	1.850	4	6	3	17	19.698	3.283
5	7	3	13	22.512	3.752	5	7	3	49	26.712	4.452
6	6	3	05	18.498	3.083	6	6	3	48	22.800	3.800
7	8	11	45	94.000	7.833	7	8	11	45	94.000	7.833
8	6	1	39	9.900	1.650	8	6	3	44	22.398	3.733
9	9	2	46	24.894	4.149	9	9	3	05	27.747	4 624
10	7	6	06	42.700	6.100	10	7	6	04	42.462	6.066
11	4	5	45	23.000	3.833	11	4	6	00	24.000	4.000
12	14	4	30	63.000	10.500	12	14	4	57	69.300	11.550

TABLE "F."

TOTAL AVERAGE LAMPS NIGHT SERVICE—6 P.M. TO 6 A.M.

Date.	Lamps. Nominal c. p.	Building.	Express Office.	Train Shed.	Total.
October 11..........	16	133.91	3.178	....	137.088
"	32	17.75	2.985	26.266	47.001
" 12..........	16	157.08	3.178	....	160.258
"	32	19.42	2.985	31.897	54.302
" 13..........	16	144.08	3.178	....	147.258
"	32	17.666	2.985	31.282	51.933
" 14..........	16	142.41	3.178	....	145.588
"	32	17.08	2.985	31.397	51.462
" 15..........	16	144.75	3.178	....	147.928
"	32	16.42	2.985	29.697	49.102
" 16..........	16	134.58	3.178	....	137.758
"	32	17.08	2.985	27.523	47.598
" 17..........	16	144.75	3.178	....	147.928
"	32	16.75	2.985	29.795	49.530
" 18..........	16	140.40	3.178	....	143.578
"	32	18.00	2.985	33.827	54.812

TABLE "G."

TOTAL AVERAGE LAMPS NIGHT SERVICE—6 P.M. TO 10 P.M.

Date.	Lamps. Nominal c. p.	Building.	Express Office.	Train Shed.	Total.
October 11..........	16	205.00	5.157	......	210.157
"	32	31.8	3.750	45.788	81.338
" 12..........	16	249.6	5.157	......	254.757
"	32	35.2	3.750	54.591	93.541
" 13..........	16	226.4	5.157	......	231.557
"	32	29.8	3.750	53.721	87.271
" 14..........	16	219.4	5.157	......	224.557
"	32	29.0	3.750	53.700	86.450
" 15..........	16	221.2	5.157	......	226.357
"	32	28.8	3.750	50.513	83.063
" 16..........	16	204.2	5.157	......	209.357
"	32	28.4	3.750	46.629	78.779
" 17..........	16	225.4	5.157	......	230.557
"	32	28.2	3.750	50.740	82.790
" 18..........	16	211.4	5.157	......	216.557
"	32	29.6	3.750	58.808	92.158

TABLE "H."

Speed of Engines—Revolutions per Minute.

Date.	Time.	East Engine.	West Engine.
October 12	Day	.. ..	233.92
" 12	Night	225.96	230.00
" 13	Day	229.60	......
" 13	Night	Counter off	233.36
" 14	Day	......	231.84
" 14	Night	228.89	Counter off.
" 15	Day	229.59	......
" 15	Night	221.62	232.47
" 16	Day	.....	231.96
" 16	Night	230.80	237.03
" 17	Day	229.86	......
" 17	Night	226.59	233.20
" 18	Day	......	235.68
" 18	Night	227.36	231.97

TABLE "J."

Comparison of Speed—Engines and Dynamos.

Date.	Engine.	Revolutions per Min. Engine.	Revolutions per Min. Dynamo.	Theoretical Rev. per Min. of Dynamo.	Percentage of Engine Speed Realized.
Oct. 12.	West, A.M.	236.358	1 172.0	1 212.99	96.62
" 13.	East, "	229.600	1 152.0	1 195.53	96.36
" 13.	West, P.M.	233.360	1 158.57	1 197.60	96.74
" 14.	" "	231.840	1 159.33	1 189.80	97.44
" 14.	East, P.M.	228.890	1 137.60	1 191.83	95.45
" 15.	" A.M.	230.000	1 145.43	1 197.61	95.64
" 15.	West, P.M.	232 470	1 162.66	1 193.03	97.45
" 15.	East, "	221.620	1 109.00	1 154.00	96.10
" 16.	West, A.M.	231.600	1 156.00	1 188.57	97.26
" 16.	" "	230.250	1 138.66	1 181.64	96.36
" 16.	East, P.M.	230.900	1 169.66	1 202.29	97.28
" 17.	" A.M.	230.460	1 145.33	1 200.00	95.44
" 17.	West, P.M.	232.57	1 165.33	1 210.99	96 23
" 17.	East, "	224.82	1 128.50	1 170.63	96.41

	Per cent.
Average, East	96.097
" West	96.870
Slip of belts, East engine	3.903
" " " West "	3.130

Note.—Fly-wheel, West Engine, covered with paper.
" East " not " " "

TABLE "K."

Indicated Horse-Power of Engines.

Date.	Time.	East Engine.	West Engine.
		Horse-Power.	Horse-Power.
October 12	Day	......	......
" 12	Night	......	......
" 13	Day	21.327	......
" 13	Night	35.975	32.888
" 14	Day	......	22.027
" 14	Night	39.701	34.008
" 15	Day	21.493	......
" 15	Night	35.863	38.076
" 16	Day	......	13.777
" 16	Night	33.425	32.586
" 17	Day	22.904	......
" 17	Night	36.377	34.624
" 18	Day	......	......
" 18	Night	......	......

TABLE "L"

Performance of Boilers.—Data.

Date.	Time Starting.	Time Stopped.	Hours Water Measured.		Tanks Used.		Total Water.	Total Coke.
					No.1.	No.2.		
	A. M.	P. M.	Hrs.	Min.			Pounds.	Pounds.
October 11.	9.25	7.17	9	52	10	10	17 502.50	2 535.00
" 12.	7.20	7.16	11	56	11	12	20 133.25	2 545.50
" 13.	7.00	5.25	10	25	8	7	13 121.50	1 974.14
" 14.	7.00	9.55	14	55	15	15	26 253.75	3 195.92
" 15.	7.00	5.00	10	00	7	7	12 251.75	1 593.93
" 16.	7.00	9.50	14	50	13	13	22 753.25	2 990.58
" 17.	8.15	5.55	9	40	8	7	13 121.50	1 973.64
	P. M.	A. M.						
" 18.	2.00	1.00	11	00	13	12	21 872.75	2 602.90

ECONOMY.—STEAM PER POUND OF COKE.

DATE.	Apparent Evaporation.	Actual Evaporation.	Temp. Feed Water.	Evaporation from and at 212° Fahr.
October 11.........	7.023	6.788	67.0	7.811
" 12.........	7.909	7.645	66.5	8.801
" 13.........	6.646	6.424	65.8	7.400
" 14.........	8.215	7.941	64.0	9.161
" 15.........	7.686	7.429	64.2	8.569
" 16.........	7.608	7.354	63.7	8.493
" 17.........	6.648	6.426	62.6	7.422
" 18.........	8.403	8.122	62.6	9.381
Average..................		7.266	....	8.380

TABLE "M."

QUALITY OF STEAM.—CALORIMETER.

Average steam condensed, pounds..........................	12.763
" water heated, "	200.00
" initial temperature, Fahr............................	62.42
" " " corrected	62.43
" final " Fahr.........................	129.34
" " " corrected.........................	129.66
Range	67.23
Heat units per pound of steam............................	1 183.174
Average steam pressure, pounds...........................	80.057
Heat units at observed pressure..........................	1 212.720
Difference	29.546
Latent heat of steam.....................................	885.590
Percentage of water entrained............................	3.336
Efficiency of steam......................................	0.97564

TABLE "N."

COMPARISON OF POWER AND LAMPS.

Date.		Time.	Horse-Power.	Lamps.		Equivalent 16 c. p. Lamps.	16 c. p. Lamps per H. P.
				16 c. p.	32 c. p.		
October	13.	Day.	21.327	81.20	9.73	100.66	*4.719
"	13.	Night.	68.863	231.557	87.271	406.099	5.897
"	14.	Day.	22.027	86.06	9.29	104.64	4.750
"	14.	Night	73.709	224.557	86.450	397.457	5.392
"	15.	Day.	21.493	85.51	11.36	108.23	5.035
"	15.	Night.	73.939	226.357	83.063	392.483	5.308
"	16.	Day.	13.777	29.86	4.85	39.56	2.871
"	16.	Night.	66.011	209.357	78.779	366.915	5.558
"	17.	Day.	22.904	79.85	10.57	100,99	4.409
"	17.	Night.	71.001	230.557	82.79	396.137	5.579

LAMPS, 16 NOMINAL C. P. PER INDICATED HORSE-POWER OF ENGINE.

Average for day (including October 16th)	4.357
" " " (omitting " ")	4.728
" " night	5.547
" " day, 24 hours, including October 16th	4.952
" " " " omitting " "	5.136

DISCUSSION.

JOHN W. HOWELL, C. E.—The Central Union Depot in Cincinnati is lighted by an Edison Isolated Electric Light plant. The plant is owned by local parties, who operate it and sell the current to the Depot Company, the Depot Company supplying steam and the owners of the plant supplying the lamps.

The contract between these parties gives the Depot Company the privilege of lighting the depot for one week of each year with gas, and if this week's lighting costs less than the following week's lighting by electric light, then the price of electric light must be reduced in like proportion.

About two years ago the Depot Company lighted their depot with gas for one week and with electric light for one week, as specified in the contract. This test was conducted by the Depot Company themselves, and the results showed that lighting by electric light cost them 40 per cent. less than lighting by gas.

In October, 1887, the Depot Company had another comparison made, the entire conducting of the test being placed in the hands of the Cincinnati Gas Company. I went to Cincinnati to look after the interest of the Electric Light Company, but was unable to secure any information as to the test proposed. The owners of the electric light plant were not recognized in this test; I therefore withdrew and watched the proceedings.

Previous to the test, the electric light plant was shut down for eight days and the depot lighted by gas; the Gas Company keeping a number of men on the watch to see that due economy was observed in the use of the gas. For the next eight days the depot was lighted by electricity, the owners of the electric plant merely seeing that the same economy as to light was observed as during the preceding test. The cost of the electric light during those eight days was 10 per cent. less than that of the gas during the previous eight days. Consequently the owners of the electric light plant were not required to reduce the price at which they furnished the light.

These tests furnished a direct comparison between the cost of lighting a given building with gas at $1 15 per 1 000 feet and with electric light. The same amount of light was required in both cases, and all conditions were the same.

I think that it is to be regretted that the comparison instituted by Mr. Hill was not based on the actual result financially of these tests, instead of on the relative illuminating power of gas and incandescent lamps.

A few words in explanation of the small number of lamps per horsepower shown by the test. The parties installing this depot plant, to

secure themselves against accident, put in two engines and counter-shafting, which enables them to use either engine to run the dynamos. This largely increases the friction, and the friction of the shaft is a practically constant quantity. Owing to the rigid economy used in burning the lamps during the test, the power measurements were taken with first, only about one-eighth load on the dynamos and second, with very little over half a load on; this not only puts the dynamos under its worst conditions of economy, but the large shaft friction being charged against a very small load, makes a very bad showing for the lamps. With the same lamps and dynamos used in Cincinnati, with a good load the Edison Company has often guaranteed eight lamps of sixteen candles for each horse-power delivered to the dynamos. This guarantee has been frequently tested and has never been disproved. The Edison Company now sells lamps and dynamos with which it guarantees twelve lamps of sixteen candles each for each horse-power delivered to the dynamos. This guarantee has been tested at two plants and in both cases the full number of lamps guaranteed was obtained. The first of these tests was made at Lowell by the company owning the water-power there, they using their own dynamometer and making the test themselves. The second test was made at the Toledo, O., State Asylum by State experts.

In both cases the results showed a small margin above the guarantee —both showed over twelve lamps per horse-power delivered to the dynamos. Mr. Hill gives two measurements of horse-power and lamps, the first one about four and three-fourth lamps to the horse-power, with one-eighth load on the dynamo, and the second five and three-fourths, with half load. Full load on the dynamos ought to get seven or more. The larger the load on the dynamo the smaller the proportion of the total load is friction and the better the result. It is against the practice of the Edison Company to use shafting between the engine and dynamos. They always belt direct from engine to dynamo; this reduces the friction to a minimum. With a full load on the dynamo the friction of the engine and dynamo will be about 20 per cent. of the total load; with one-eighth of the load I would expect 40 or 50 per cent. of the total to be friction. In an engine running with one-eighth of its load what would the friction be? What per cent. of the total?

Mr. Hill notes a very great loss of energy between the engine and the electric light circuit, and it is due to this loss that the number of lamps per horse-power is so small. This loss is made up of two parts, the loss in the dynamo and friction. There have been a great many tests of dynamos published, all of which show the losses in dynamos to be very small. Losses by friction in engines and shafting are exceedingly variable, and depend entirely upon the condition of the bearings at the time. Mr. Hill assumes the loss by friction, and by inference charges the remaining loss to the dynamo. Considering the very great

uncertainty of losses by friction, and the ease with which it could have been measured, the assumption seemed decidedly unscientific.

There is one other point. Where Mr. Hill speaks of the great variation in the light given by incandescent lamps, when making measurements of the candle light of the lamps, he gives the variation as 20 per cent., and he does not attempt to give any reason for it. There is only one thing that can make the lights vary, and that is the variation of the pressure on the line, and there are very few things that will make that vary; either very great changes in the load or in the speed of the engine. I am very well satisfied that the changes in the lights were due to changes in the speed of the engine. Any change in the speed of an engine will make the lights go up or down, as the case may be. When making measurements, if there are sudden changes it is undoubtedly due to changes in the speed of the engine. We all know that an electric light is steady; we know that its steadiness depends upon the steadiness of the speed of the dynamo. Latterly electric lighting has developed a type of engines that give better results than ever before. It is necessary in electric lighting to have a steady speed, and I think the alarm over the danger of variation in the electric light is not at all warranted. I think that in a large majority of cases they give a very steady, uniform light.

As a lamp gets older it takes less power to run it, but it gives less light; the reduction in light is faster than the reduction in power necessary to run it; it takes more power per unit of light as the lamp gets older, but the lamp requires less power to run it.

John W. Hill, M. Am. Soc. C. E.—The writer was engaged to make the investigation by the executive officer of the Depot Company, and was informed, prior to beginning work, that his selection was reported to and approved by the officers of the Electric Company. If Mr. Howell, as he states, was present during the test, the writer can only regret that he did not make himself known and give him (writer) the benefit of his knowledge and experience in these matters.

All of the data touching the cost of producing the light by the Edison system were taken jointly by one set of observers under direction of the writer, and another set under direction of the Electric Lighting Company, and he is not aware that any exception by the Electric Lighting Company has ever been taken either to the methods employed in making the test or in the accuracy of the data as reported.

While it is true that the distribution of the indicated horse-power of the engines is not as accurate as it could have been by means of special tests (before or after the general tests) for friction of engine, jack shaft and dynamos—which tests, in passing, it is well to remark, were not admissible, owing to circumstances beyond my control—the main elements of the question, however, were carefully measured, *i. e.*, the power developed by the engines and the electrical energy developed in the lamps.

The steam engine indicators were in perfect working order, and the springs carefully tested for these experiments. The assistants in charge of the engine test were experienced and careful men. The motion of the engines was taken with reliable, continuous counters, Schaffer & Budenberg make. The apparatus for the photometric tests was well constructed and worked by Professor French, with extreme accuracy in view; while the galvanometer was said to be one of the most perfect instruments in the country, and the readings were taken by a skillful electrician after a careful calibration in the test circuit to reconcile the effect of local disturbances.

While it may be true that more of the loss between the engines and lamps should be charged to the engine and connectors and less to the current generator, the main fact which I wished to emphasize remains unchanged, viz.: of the indicated power of the engines, which power is a direct and exact measure of the cost of producing the light, only 40 per cent. is realized in the illuminating effect of the incandescent lamps.

It would no doubt be very interesting to know how this loss of about 30 per cent. is divided up, but no opportunity was afforded to do so in this test.

The frictional loss in modern automatic engines and jack and line shafts has been so fully investigated by experimenters (the writer among others), that no serious mistake should be made in apportioning the power so expended under known conditions of service.

The plates of the zinc voltameters were weighed by an employee of the Edison Light Company in my presence, and his determinations accepted as correct.

The tests for quality of gas furnished by the Cincinnati Gas Company were made by Professor French of the Cincinnati University; and the tests for brilliancy of the electric lamps were made by Professor French, Mr. C. E. Jones (a local electrician), and the writer, and I am quite sure the prejudices of these gentlemen (if they had any) were in favor of the electric light.

There was a test with gas for eight days with the following results:

Cubic feet of gas consumed	214 700	
Cost at $1 15 per 1 000 cubic feet		$246 90
Average actual candle-power of the gas, for a consumption per hour of five cubic feet, as measured by Professor French	16 94	
Average lights of 16 candle-power per hour for eight days	236 81	
Cost per light of 16 actual candle-power for eight days		1 04

From the test of the electric lighting plant, the data of which is all in the writer's possession, and for the accuracy and fairness of which

he personally vouches, there were as average effects for the eight days of the test, the following:

Cost of lights as per the rate paid by the Union Depot Company to the Edison Electric Light Company (not including the cost of furnishing the steam to operate the driving engines)	$167 00
Cost of the steam, including all items of expense, which is borne by the Depot Company	101 32
Total cost for eight days	$268 32
Average hourly illumination by the electric light plant in lights of 16 actual candle-power each for eight days	185 15
Cost per light for eight days	1 45

From which, by the simple rule of three, is deducted the increased cost of electric lighting over gas lighting of more than 39 per cent.

The comparative cost of the two distinctive modes of lighting was not based on theory.

The only facts in connection with gas which it is necessary to know to institute a comparison of cost, for equal effects of illumination, are the candle-power of the gas (which in this instance, as has been stated in the paper, was carefully measured by Professor French), and the cost of the gas per thousand cubic feet, which is $1.15 net. Having these data, the comparison of cost can be made practically.

The writer was not present during the test of lighting the Depot Building with gas, and the quantity consumed during the eight days was communicated to him by the President of the Gas Company, General A. Hickenlooper, and the quality of the gas furnished by the Gas Company during the test of eight days was reported to him by Professor French, and he has no reason to question the accuracy of either. But assuming that the statement of the quantity of gas consumed is incorrect, this has no influence on the comparison of costs; the value of the gas, in candle-power, and its cost per 1 000 cubic feet, are all the data required to make such a comparison. Taking the actual data of the eight days' test with gas, and comparing with the data given in the paper, we have by a different method of stating the case the same relative results.

In regard to the superfluity of shafting, of which Mr Howell complains, there is one counter shaft between the engines and the dynamos (and I think this is the usual arrangement employed with similar plants), to increase the speed from 230 at the engine to 1 150 at the dynamos.

The engines were built by the Buckeye Engine Company, of Salem, O., which concern has had considerable experience in constructing engines for electric lighting plants, and these were rated at a maximum

capacity of 50 indicated horse-power each, and were operated during the test of eight days with an average load of 30.34 indicated horse-power each, or at three-fifths of the rated capacity, and not one-eighth, as Mr. Howell put it, the load on the engines being five times his assumption. The cylinders are 10 inches diameter, 14 inches stroke, and the engines, as stated above, operated at a nominal speed of 230 revolutions per minute.

The fluctuations in candle-power of the lamps while under photometric test, I supposed at the time, were due to variations in the speed of the engines driving the dynamos, but as this variation seems to be unavoidable with driving engines as at present constructed, I cannot see how an apology for the defects of the engine is calculated to diminish this vicious and altogether objectionable feature of incandescent lighting. As to the merit of the writer's opinion on the injurious effect of an unsteady light or of rapid fluctuations and wide range of candle-power in any lamp upon the organs of vision, he who doubts its correctness is respectfully referred to any experienced occulist. I have taken the liberty of asking General Hickenlooper to give me a written statement of his knowledge of the motive of this investigation, and to correct or explain such matters in the discussion as were calculated to impair the scientific value of the facts presented in the paper.

The substance of his reply is appended as part of this discussion.

A. Hickenlooper.—Some of the statements by John W. Howell with reference to the competitive test of gas and electric lighting at the Grand Central Depot in Cincinnati, do not accord with my recollection of the facts.

First.—As regards the statement that the Depot Company made a test two years ago, and found that lighting by electricity cost 40 per cent. less than lighting by gas.

The fact is that the Electric Light Company made the test, and neither the Gas Company nor the Depot Company had anything to do with it. The result of the test as published by the Electric Light Company was that, on an eight night test, the gas cost $497 28, and the electric light $343 29. This test was made during the eight longest nights of the year.

At the same rate per night, this would make gas lighting for the year cost $22 703 94, but in fact the Gas Company did light the depot for one year at a cost of $5 892.

The cost of the electric light last year was $14 664 12.

The statement that the last test in October, 1887, was made entirely by the Gas Company, is not correct. It was entrusted to the experts, John W. Hill and Professor Thomas French.

As to the cost of the last test, the records show that for electric lighting the cost was—

For steam service..............................	$102 03
The meters showed a consumption at contract rate of..........	167 00
Total..	$269 03

while the same degree of illumination by gas required the use of 167 801 cubic feet, at $1 15 = $192 97.

The total gas used on the premises during the gas test, was 196 700 cubic feet, at $1 15 = $226 20.

The conclusions regarding relative cost were not arrived at by taking the theoretical cost in either case. The actual cost of lighting by both gas and electricity was used.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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(Vol. XVIII.—June, 1888.)

FORMULAS FOR THE WEIGHTS OF BRIDGE TRUSSES.

By A. J. Du Bois, Jun. Am. Soc. C. E.

Presented at the Annual Convention, July 2d, 1888.

In the Society Transactions for May, 1887, Vol. XVI, page 191, the writer presented a series of rational formulas for plate girders and truss spans, which gave rise to valuable discussion.

As regards the formula for trusses then presented, the results of the discussion may be fairly summed up as follows :

First.—The formula presented, was considered as too complicated, required too much labor to use, and was not sufficiently accurate to justify its use as a guide in practice, or as a basis of estimates of cost.

Second.—The method by which the strut formula was introduced was criticised as unsound.

Third.—It was objected that no proper provision was made for details, such as lattice bars, pins, eye-bar heads, etc.

Fourth.—It was asserted that a general rational formula which should include all the styles of truss in use, and while giving accurate results still be sufficiently simple for practical use, was in the nature of things, impossible.

While, therefore, the formula was very generally admitted to be valuable, and more accurate than any previous formula making claim to

rationality, the general opinion seemed to be that it was itself so largely empiric, as to justify its being classed as such. Therefore theoretic deductions from it could not be safely trusted, while its accuracy was not sufficient to pay for the trouble of using it in cases where a close estimate was desired. Hence a simpler empiric formula would better answer every practical purpose.

Now from the last conclusion, that a general rational formula, accurate and sufficiently simple, is impossible, I expressed then, and wish to express now, strong dissent. I was able to see no reason why this problem, which is daily solved in particular cases, should not admit of a general solution, and as no such reason was given by any one in the discussion referred to, I am still unable to conceive upon what grounds the attempt is considered beforehand as a hopeless one, unless, indeed, it be the conceded fact that no one has thus far succeeded in it.

The outline of the method of attack I adopted was as follows:

The strain in each member of the truss was found in general terms, for any depth, d, span, l, number of panels, N, and loading, precisely in the same way as in any special case. The unit stress, modified for struts as usual, was then introduced and the area of each member found. The introduction of the strut formula here was one of the points called in question. The area of each member thus found multiplied by the length, gave the contents, and the weight was thus easily found. My formula contained but two empiric constants and gave good results over a wide range.

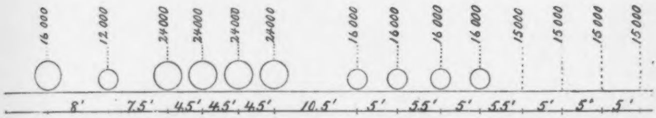
Still the first three objections which I have enumerated have caused me to devote further attention and much labor to the problem, and the belief that I have successfully met them is my excuse for this paper.

I believe that the perfected formula which I now wish to present, practically meets all the objections noted. It is thoroughly rational, simple, and easy to use, general in application to different styles of truss, accurate enough for an estimate, and hence of value in deciding between different styles, dimensions, etc. I have inserted the strut formula in a manner which seems free from objection and have taken proper account of details and secondary bracing. These are large claims, and I make them plainly in order to invite criticism and engage the labor of others in order to substantiate them, if true, or expose them if false. The great value such a formula would have, if these claims stand the test, is beyond question; and that they will therefore be very thoroughly tested I have no doubt.

LIVE LOAD.—The first step in the problem is to assume the live load for which the truss is to be designed. I assume two "typical locomotives" with tenders, followed by a train load of 3 000 pounds per foot, as given by Mr. Pegram in his paper, Transactions for February, 1886, Volume XV.

I assume this train load to be applied by concentrated loads of 15 000 pounds at every five feet.

Of course any live load desired may be assumed. I take the preceding, both for purposes of illustration and because I believe it to be such a loading as meets the requirements of present and future practice. A sketch of this loading is given here.



In determining the strains in the members, I have found that it leads to too much complication to work out the strains due to live and dead loads separately. In order to avoid such complication, I have taken the strains for a uniform load only. This, while correct enough for the flanges gives too little for the web. In order to meet this defect I suppose the truss covered with an equivalent uniform load, that is, such a uniform load as would cause at the center of the span a moment equal to the greatest moment at that point due to the actual assumed load system.

I denote this equivalent uniform load per foot per truss by w_1.

Our first step then is to make out, once for all, a table giving the value of w_1 for our assumed load system, for different spans. This is easily done for any desired system. In the table which follows we give the values of w_1 for the two typical locomotives and train load already described. Those wishing to use the formula for any other system of loads must of course make out such a table for the system assumed. It is only necessary to find the maximum moment at the center in foot pounds, and divide eight times this moment by the square of the span in feet. The excellent method by slide and diagram, described by J. E. Greiner, C. E., in the *Engineering News*, April 14th, 1888, affords a swift and accurate means of finding the maximum center moment for any system of loads whatever.

TABLE No. 1.

Equivalent Uniform Load w_1, per Foot per Truss, on the Basis of Assumed Load System, for Single Track. For Double Track, take Double Values. Span in Feet, w_1 in Pounds.

Span =	50	55	60	65	70	75	80	90	100	110
w_1 =	1 848	1 733	1 718	1 677	1 618	1 570	1 546	1 552	1 560	1 569
Span =	120	130	140	150	160	170	180	190		
w_1 =	1 579	1 586	1 574	1 562	1 556	1 546	1 533	1 516		
Span =	200	210	220	230 feet and over.						
w_1 =	1 511	1 506	1 502	1 500 pounds.						

The table gives w_1 for one truss, on the assumption of two trusses to the bridge.

Dead Load.—Our next step is to find all the dead load exterior to the truss itself. This consists of the rails, ties, planking, spikes, etc., of the cross-girders and stringers, and of the wind bracing.

All these quantities are independent of style of truss, and are capable of accurate and independent estimate. For railway bridges, a usual estimate for rails, ties, planking, spikes, etc., is 400 pounds per foot, or 200 pounds per foot per truss, if there are two trusses. For highway bridges, a separate estimate must be made for each case, according to circumstances.

The weight of cross-girders and stringers can be easily estimated for any assumed live load and panel length whatever may be the style of truss. Let us call the weight per foot per truss of rails, ties, etc., and of cross-girders and stringers, w_2. A table for w_2 can easily be filled out once for all. We give such a table for illustration.

Any designer or bridge company has the materials at hand for filling out such a table according to their own practice. The table we give here has already been given in our previous paper.

TABLE No. 2.

FOR RAILWAY BRIDGES.

Weight w_2 per Foot per Truss, of Floor, Stringers and Cross-Girders, for Single and Double Track, for Assumed Load System. Rails, Ties, etc., taken at 200 pounds per Foot per Truss. Iron Plate Girders for Stringers and Cross-Girders.

Panel Length in Feet.	SINGLE TRACK—15 FEET WIDE.				DOUBLE TRACK—25 FEET WIDE.			
	Weight of One-half Cross-Girder.	Weight of One Stringer.	Rails, Ties, etc., at 200.	w_2	Weight of One-half Cross-Girder.	Weight of One Stringer.	Rails, Ties, etc., at 200.	w_2
5	507	188	1 000	339	1 555	376	2 000	786
6	582	248	1 200	338	1 783	496	2 400	780
7	631	315	1 400	335	1 935	630	2 800	766
8	679	386	1 600	333	1 992	772	3 200	745
9	695	463	1 800	327	2 135	926	3 600	740
10	716	545	2 000	326	2 192	1 090	4 000	728
11	750	657	2 200	328	2 291	1 314	4 400	727
12	776	825	2 400	333	2 374	1 650	4 800	735
13	806	974	2 600	337	2 443	1 948	5 200	738
14	831	1 130	2 800	340	2 540	2 260	5 600	743
15	853	1 292	3 000	343	2 606	2 584	6 000	746
16	872	1 460	3 200	345	2 664	2 920	6 400	748
17	886	1 634	3 400	348	2 700	3 268	6 800	751
18	906	1 816	3 600	351	2 775	3 632	7 200	755
19	930	2 003	3 800	354	2 841	4 006	7 600	760
20	949	2 197	4 000	357	2 871	4 394	8 000	763
21	968	2 420	4 200	361	2 957	4 840	8 400	771
22	983	2 652	4 400	365	3 004	5 304	8 800	777
23	1 001	2 900	4 600	369	3 030	5 800	9 200	783
24	1 018	3 133	4 800	373	3 080	6 266	9 600	789
25	1 034	3 382	5 000	376	3 128	6 764	10 000	795
26	1 048	3 633	5 200	380	3 172	7 266	10 400	801
27	1 062	3 904	5 400	384	3 213	7 808	10 800	808
28	1 077	4 180	5 600	387	3 282	8 360	11 200	816
29	1 086	4 463	5 800	391	3 332	8 926	11 600	822
30	1 104	4 756	6 000	395	3 373	9 512	12 000	829

For other widths than those assumed in the table, we may easily change the weight of one-half cross-girder, which is the only item affected, by multiplying by $\frac{\text{width}}{15}$ or $\frac{\text{width}}{25}$.

So also the values for rails, ties, etc., may be changed to correspond to any special case, if the allowance of 200 pounds per foot is not satisfactory. If the stringers or cross-girders, or both, are of wood, the corresponding values can easily be found. In short, it is, I think, evident, that tables giving w_2 accurately, may easily be drawn up, in accordance with any specifications and any variety of circumstances. Our Table No. 2, will serve not only as an illustration of what is meant, but also will, we believe, give good values of w_2 for standard specifications, and best practice of to-day.

We have thus far, then, found and tabulated w_1 and w_2. It remains to estimate the weight of wind bracing, w_3. Here, again, tables should be drawn up giving the actual results of practice. In place of such tables we venture to submit the following simple empiric formulas, which we have found quite accurate over a wide range of cases. We do not propose these formulas as better than the tables alluded to. The actual results of practice are, of course, preferable. The formulas for w_3 are as follows:

Formulas for Estimating Weight per Foot per Truss of Wind Bracing for Railway and Highway Bridges.

For pony trusses, depth below 12.5 feet:

$$w_3 = 1.8\, N + \frac{270}{p}$$

Through trusses, upper and lower horizontal bracing only. Depth between 12.5 and 24 feet:

$$w_3 = 3.2\, N + \frac{336}{p}$$

For through or deck trusses, with vertical sway bracing, depth above 24 feet:

$$w_3 = \frac{3\, Nl}{170} + \frac{568}{p}$$

N = number of panels; l = span in feet; p = panel length in feet.

We have thus far, we trust, made it clear that all the dead load, exclusive of the truss itself, admits of easy and accurate determination, without any calculation whatever, by simple inspection of properly prepared tables, which can be drawn up once for all, independent of the truss. This external load admits of such endless variation, especially in the case of highway bridges, that it would be, indeed, a hopeless task to attempt to include it in any rational formula. It is, we believe, just this attempt which has rendered all such formulas heretofore worthless.

For railway bridges we have, then, for dead load, two tables and a set of formulas for wind bracing. It is open to any one who takes exception to either to replace them by others. We believe the values given will be found accurate enough for estimates upon the basis of any of the best standard specifications.

Weight of Truss.—Demonstration of Formula.

We now finally arrive at the truss itself. Let the weight per foot of the truss be w_4 pounds per foot, due to the actually supporting members, and let w_0 be the weight per foot of the lattice bars, pins, eye-bar heads,

splice plates, etc.; in short, everything not actually supporting. It is evident that if we can find w and w_0 accurately, the problem is solved. Thus, the total weight of iron in the bridge, exclusive of course of shoes, roller plates, etc., is $2\,(w_2 - 200 + w_3 + w_4 + w_0)$. We have already disposed of w_2 and w_3. The 200 subtracted is for the rails, ties, etc., included in w_2.

Let us take, as an illustration of our method of dealing with the truss, a Warren girder of span l, and panel length p, load on bottom chord as the simplest case. Let the number of panels be N. The load at each lower panel point is then $(w_1 + w_2 + w_3 + w_4 + w_0)\,p$, where w_1 is the equivalent live load per foot; w_2, the floor; w_3, the wind bracing; w_4, the truss proper; w_0, the details, all per foot per truss. For brevity let us call this $W p$.

The end reaction is then $\dfrac{W(N-1)\,p}{2}$.

The strain in the first lower panel is the reaction multiplied by $\dfrac{p}{2}$ and divided by the depth, d. If this strain is divided by the stress per square inch for tension R_t, we have the area. The area multiplied by p gives the volume, and this by $\frac{10}{3}$ gives the weight for iron. We have thus for the weight of the first lower panel

$$\frac{10\;Wp^3}{12\,R_t\;d}\,[N-1]$$

In precisely similar way we find the weight of each lower panel, as follows:

Weight of first lower panel,	$\dfrac{10\;W p^3}{12\;R_t\;d}$	$[N-1]$
" " second "	"	$[3\,(N-1) - 2]$
" " third "	"	$[5\,(N-1) - 8]$
" " fourth "	"	$[7\,(N-1) - 18]$

Summing up the series we have for the weight of the whole lower chord,

$$\frac{5\;WNp^3\,(N^2-1)}{18\;R_t\;d}$$

Dividing by $l = Np$, we have the weight per foot $\dfrac{5\;Wp^2\,(N^2-1)}{18\;R_t\;d}$. In precisely similar manner we have for the upper chord the weight per foot per truss $\dfrac{5\;Wp^2\,(N^2-1)}{18\;R_c\;d}$, where R_c is the unit stress for compression.

For the ties, in like manner, we easily find $\dfrac{5\;W\,(p^2 + 4\;d^2)\;N}{24\;R_t\;d}$, and for the struts $\dfrac{5\;W\,(p^2 + 4\;d^2)\;N}{24\;R_c\;d}$.

Adding these four quantities we have for the total weight per foot of one truss, exclusive of the details, w_4,

$$w_4 = \frac{5\,W}{18\,d}\Big[\frac{(N^2-1)\,p^2}{R_t} + \frac{(N^2-1)\,p^2}{R_c} + \frac{0.75\,(p^2+4\,d^2)\,N}{R_t} + \frac{0.75\,(p^2+4\,d^2)\,N}{R_c}\Big]$$

For the sake of brevity let us put $T = (N^2-1)\,p^2 + 0.75\,N\,(p^2+4\,d^2)$ $C = (N^2-1)\,p^2$, $S = 0.75\,N\,(p^2+4\,d^2)$, where T refers to the tension members, C to the upper chords, and S to the struts.

Placing for W its value, we have now

$$w_4 = \frac{5\,(w_1+w_2+w_3+w_4+w_0)}{18\,d}\Big[\frac{T}{R_t}+\frac{C}{R_c}+\frac{S}{R_s}\Big]$$

From this we have

$$w_4 = \frac{\dfrac{w_1+w_2+w_3+w_0}{3.6\,d}}{\dfrac{T}{R_t}+\dfrac{C}{R_c}+\dfrac{S}{R_s}} - 1$$

Thus far, it will be observed, everything is entirely rational. We have, it is true, considered the truss as uniformly loaded with the full load. This gives somewhat too little for the ties and struts, the maximum strain in which is caused by a varying position of the live load. To balance this we have taken w_1 as the equivalent uniform load, thereby getting somewhat too much for the flanges.

Now the strut formula for the upper chords may be written $R_c = \dfrac{\mu}{1+\dfrac{p^2}{250\,r_1^2}}$, and for the struts, $R_s = \dfrac{\mu}{1+\dfrac{p^2+4\,d^2}{4\times 125\,r_2^2}}$, p and d being in feet. The numerator μ is usually taken for iron at 8 000.

Inserting these values of R_c and R_s, we have the weight of the truss alone, exclusive of details,

$$w_4 = \frac{\dfrac{w_1+w_2+w_3+w_0}{3.6\,\mu d}}{\dfrac{\mu}{R_t}T + C + S + \dfrac{Cp^2}{250\,r_1^2} + \dfrac{S\,(p^2+4\,d^2)}{500\,r_2^2}} - 1$$

Now the unit stress for tension R_t is on the average about the same as μ. We can without appreciable error, therefore, put $\dfrac{\mu}{R_t} = 1$.

For the square of the radius of gyration for the chords, r_1^2, I find that the simple expression $\frac{(N-1)\,p^2}{100}$ gives very close values when compared with actual practice. For the struts we take $\frac{(N-1)}{50}$ multiplied by the square of the length, or in this case, $\frac{(N-1)\,(p^2+4d^2)}{200}$. This is our first introduction of anything empirical. These simple expressions give, we believe, the values of r_1^2 and r_2^2 with all desirable accuracy.

Inserting them we have

$$w_4 = \frac{\dfrac{w_1 + w_2 + w_3 + w_0}{3.6\,\mu d}}{\dfrac{}{\alpha p^2 + \beta d^2 + \dfrac{\alpha p^2 + 4\,\beta d^2}{5\,(N-1)}} - 1} \quad \ldots\ldots\ldots(1)$$

where $\alpha = (2\,N^2 + 1.5\,N - 2)$ and $\beta = 6\,N$ for Warren girder.

For single intersection Pratt truss, we should have

$$\alpha = (2\,N^2 + 3\,N - 2) \text{ and } \beta = 3\left(2\,N - 4 + \frac{11}{N}\right)$$

For double intersection Whipple truss,

$$\alpha = 2\left(N^2 + 3\,N - 10 + \frac{12}{N}\right) \text{ and } \beta = 3\left(N - 2 + \frac{16}{N}\right)$$

For any style of truss the values of α and β are easily made out once for all. They are the co-efficients of p^2 and d^2 given in our previous paper for the value of the quantity we there called "A." A comparison of (1) with the formula for w_4 given in that paper, will show a marked difference in two respects. First, w_0 did not occur in that formula. Second, in place of the term

$$\frac{\alpha\,p^2 + 4\,\beta\,d^2}{5(N-1)} \text{ we had } \frac{45\,p^2 + 202\,d^2}{(w_1 + w_2 + w_3)\,p}$$

It will be seen that we now have varying constants α and $4\,\beta$, in place of 45 and 202, and the details are now included in w_0.

Here, then, we have in (1) a formula, entirely rational, made out by the same statical principles which would be employed in the solution of any special case, and following step by step the method of special calculation. We have also introduced the strut formula, thereby allowing for the increase of section required by long struts. It only remains to find the value of w_0, or the weight per foot of lattice bars, eye-bar heads, pins, splice plates, etc.

VALUE OF w_0.—We have thus far made no attempt to fit our formula to any special cases, the only empiricism being in the introduction of the radius of gyration. In the discussion upon my previous paper Mr.

Pegram gave the weights of a 255.5-foot span for different depths and panel length.

Making use of his data we have applied formula (1) and determined what w_0 should be, provided the formula is in all other respects correct. The results are as follows:

For $N=14$, $p=18.25$

$d=$ 29	32	35	38
$w_0=$ 118	132	150	164

For $N=10$, $p=25.55$

$d=$ 29	32	35	38
$w_0=$ 116	126	136	145

It will be observed that in the first case, the difference is 14 pounds for every three feet of depth. The panel weight for one foot of depth, or the weight of details per foot of depth is then $\frac{14}{3} \times 18.25 = 85.16$ pounds.

In the second case we have a difference of 10 pounds for every 3 feet of depth. The weight of details per foot of depth is then $\frac{10}{3} \times 25.55 =$ 85.16 pounds, or precisely the same, just as it should be.

Does not this indicate that the formula is correct?

Now from this case of 255.5-foot span alone I deduce for w_0 the empirical formula $w_0 = \frac{Nd}{3} + A$

where A depends only upon the number of panels, and is given by

$$A = 0.875\,N\,(12-N)+6$$

This last value can be tabulated, and we thus have the following table:

TABLE No. 3.

$$w_0 = \frac{Nd}{3} + A.$$

$N=$	4	5	6	7	8	9	10	11	12	13
$A=$	34	36.6	37.5	36.6	34	29.6	23.5	15.6	6	—5.37

$N=$	14	15	16	17	18	19	20
$A=$	—18.5	—33.4	—50	—68.4	—88.5	—110.4	—134

Our values of α and β also can be tabulated as follows:

TABLE No. 4.

VALUES OF α AND β.

N	Single intersection.		Double intersection.		Warren.	
	α	β	α	β	α	β
2	12	16.5	12	24	9	12
3	25	17	24	19	20.5	18
4	42	20.25	42	18	36	24
5	63	23.6	64.8	18.6	55.5	30
6	88	29.5	92	20	79	36
7	117	34.714	123.428	21.857	106.5	42
8	150	40.125	159	24	138	48
9	187	45.666	198.666	26.333	173.5	54
10	228	51.3	242.4	28.8	213	60
11	273	57	290.1818	31.3636	266.5	66
12	322	62.666	342	34	304	72
13	375	68.538	397.846	36.69	355.5	78
14	432	74.357	457.714	39.4285	411	84
15	493	80.2	521.6	42.2	470.5	90
16	558	86.0626	589.5	45	534	96
17	627	91.941	661.412	47.8235	601.5	102
18	700	97.833	737.333	50.666	673	108
19	777	103.737	817.263	53.5263	748.5	114
20	858	109.65	901.4	56.4	8.28	120

RESULT OF DEMONSTRATION.

We may sum up the results of our discussion then, as follows:

Total weight of iron in pounds per foot $= 2\ (w_2 - 200 + w_3 + w_4 + w_0) \ldots$ I

Where w_2 is the weight per foot per truss of cross-girders, stringers and rails, ties, etc., to be taken at once from a table similar to Table No. 2.

We have given simple formulas for the weight per foot per truss of wind bracing w_3, or it may also be taken out directly from a table embodying the results of actual practice.

Finally we have for the weight per foot per truss of the truss itself, including details,

$$\text{Weight of truss} = w_4 + w_0 = \frac{(w_1 + w_2 + w_3) + L w_0}{L - 1} \ldots\ldots\ldots\ldots \text{II}$$

Where the equivalent uniform load per foot per truss w_1, is taken directly from Table No. 1, and w_0 is given by Table No. 3.

$$\text{Finally, } L = \frac{3.6\,\mu d}{\alpha p^2 + \beta d^2 + \dfrac{\alpha p^2 + 4\,\beta d^2}{5\,(N-1)}}$$

Here μ is the numerator of the strut formula, and α and β are taken directly from Table 4.

The only empiricism is in the last term in the denominator of L and in the value of w_0. Both of these quantities are subsidiary and small with respect to the others, while the entire formula is rational in form, and has been fitted in its subsidiary empirical portions to one case only

of Mr. Pegram's examples. If such a formula thus found and fitted, checks well with practice through a wide range, it seems to me it is all I have claimed for it. The amount of calculation required is very small and requires little time when tables are used. It is simple, general and rational. The strut formula is, I think, properly introduced, and the details are taken into account. Is it accurate enough?

ACCURACY OF FORMULA.—This question can only be answered by comparison of the results with actual examples. It is sufficient to thus check II, for if this is accurate, there is no difficulty as we have seen, in knowing w_2 and w_3 directly from tables without calculation at all. The truss is the problem.

In checking II by actual examples, the actual data must be used, not those given by my tables and formulas. Those are intended for design, not for testing executed examples.

Mr. Pegram has furnished in previous discussions an extensive list of single track spans with all necessary data. He has given for each case the weight of truss $w_0 + w_4$, also the total weight of iron and weight of shoes, etc. It is therefore possible to calculate for the load systems he specifies, the value of w_1. He has also given $w_1 + w_2 + w_3$. We are unable to check these values of $w_1 + w_2 + w_3$ without manifest inconsistencies in the value of the rails, ties, etc., which we have taken at 200 pounds. Referring to I, we see at once, that if in any case we take half the total weight, subtract the weight of shoes, etc., which is known for each truss, and divide by the span, we have the weight per foot per truss. Subtract from this the known value of $w_4 + w_0$, and we have $w_2 - 200 + w_3$. If to this we add w_1 and the allowance for rails, ties, etc., we ought to have $w_1 + w_2 + w_3$.

Taking Mr. Pegram's data, I have the following tabulation:

1 Span.	2 $w_2 - 200 + w_3$ as given by Pegram.	3 w_1 as calculated.	4 $w_1 + w_2 - 200 + w_3$	5 $w_1 + w_2 + w_3$ allowing 200 for rails, etc.	6 $w_1 + w_2 + w_3$ as given by Pegram.	7 Allowance for rails, etc., from Pegram's data.
	190	1 378	1 568	1 768	1 820	252
104	200	1 492	1 692	1 892	2 000	308
	207	1 566	1 773	1 973	2 020	247
	194	1 287	1 481	1 681	1 675	194
150	200	1 470	1 670	1 870	1 844	174
	215	1 589	1 804	2 004	1 950	146
	224	1 172	1 396	1 596	1 565	169
201.5	228	1 404	1 632	1 832	1 776	144
	241	1 542	1 783	1 983	1 946	163
	347	1 027	1 374	1 574	1 605	231
320	358	1 160	1 518	1 718	1 798	280
	364	1 515	1 879	2 079	2 088	209
255.5	217	1 526	1 743	1 943	1 943	200

I am unable to account for these discrepancies. The values of w_1 have been carefully calculated. The data attributed to Mr. Pegram are taken from his published figures, and there may perhaps be clerical errors which I cannot correct. In the last case it will be observed we agree exactly. In the third case of 320-foot span we also agree, but in the other two cases of the same span, if Mr. Pegram's values are correct and my values of w_2, are also correct, we have 280 and 231 for the same allowance.

As there is, I think, no error in the values of the second column, and I consider those in the third correct, I am, I think, justified in taking for the test of my formula the values of column five, instead of those given by Mr. Pegram. I am, of course, liable to correction, if I have misunderstood Mr. Pegram's data, but it is important that the value of $w_1 + w_2 + w_3$ should be accurately found if the formula is to be truly tested.

We have then the following test of the formula; taking $\mu = 8\ 000$:

Span in feet l	Depth in feet d	No. of panels N	$w_1 + w_2 + w_3$	$\alpha p_2^2 + \beta d^2$	$\frac{\alpha p^2 + 4\beta d^2}{5(N-1)}$	L	w_0	w_4	Actual value w_4	Per cent. of difference.
			1 768					221	227	$-2\frac{55}{100}$
S. I. 104.	24	6	1 892	43 392	3 268	14.6546	86	231	235	$-1\frac{77}{100}$
			1 974					237	241	$-1\frac{75}{100}$
			1 681					311	307	$+1\frac{30}{100}$
D. I. 150.	25	9	1 870	71 641	2 956	9.6518	105	333	331	$+0\frac{60}{100}$
			2 004					348	351	$-0\frac{85}{100}$
			1 596					437	422	$+3\frac{55}{100}$
D. I. 201.5	28	12	1 832	123 100	3 692	6.36	118	481	465	$+3\frac{44}{100}$
			1 983					509	497	$+2\frac{41}{100}$
			1 574					863	855	$+0\frac{93}{100}$
D. I. 320.	20	16	1 718	287 820	5 918	3.3335	132	925	936	$-1\frac{17}{100}$
			2 079					1 080	1 100	$-1\frac{81}{100}$
	29		1 945	185 577	4 382	4.3966	118	725	726	$-0\frac{13}{100}$
	32	14	1 955	192 793	4 828	4.6634	132	701	702	$-0\frac{14}{100}$
	35		1 956	200 718	5 318	4.8923	146	686	692	$-0\frac{86}{100}$
D. I. 255.5	38		1 959	209 353	5 848	5.0854	160	678	684	$-0\frac{87}{100}$
	29		1 954	183 756	4 900	4.4271	122	727	730	$-0\frac{41}{100}$
	32	12	1 964	189 978	5 353	4.7181	134	698	702	$-0\frac{57}{100}$
	35		1 965	198 812	5 850	4.9251	146	683	682	$+0\frac{14}{100}$
	38		1 966	204 258	6 392	5.1953	158	664	670	$-0\frac{89}{100}$
	29		1 978	182 460	5 667	4.4395	121	731	715	$+2\frac{23}{100}$
	32	10	1 976	187 730	6 137	4.7537	131	692	686	$+0\frac{87}{100}$
	35		1 979	193 519	6 652	5.0356	141	666	661	$+0\frac{75}{100}$
	38		1 981	199 826	7 213	5.2859	151	648	642	$+0\frac{93}{100}$

It will, I think, be admitted that a formula put together as this one has been, employing empiric values only in two subsidiary terms, ra-

tional throughout in form, which follows step by step the main features of actual design, and which has been fitted in its empirical portion, only to the last case of the 255.5-foot span—must be correct in all important particulars, in order to give such close results over such a wide range.

In every case it gives results close enough for a bid, and it follows change of depth in the last examples with such exactness, that it is reasonable to conclude that it will give with equal exactness the result arising from any change in any of the dimensions. Any one who will refer to Table No. 4, and taking the value of $w_1 + w_2 + w_3$ as given, check any one of these cases, will be satisfied that the formula requires but little time and calculation.

Lest it may be suspected that our alteration of the values of $w_1 + w_2 + w_3$ as given by Mr. Pegram is in the interest of correspondence, we give below the results of using Mr. Pegram's own values.

Span.	$w_1 + w_2 + w_3$ as given by Pegram.	w_4 by Formula.	w_4 Actual Value.	Per cent. of Difference.
	1 820	225	227	$-0\frac{88}{100}$
104	2 000	238	235	$+1\frac{28}{100}$
	2 090	245	241	$+1\frac{66}{100}$
	1 675	310	307	$+0\frac{97}{100}$
150	1 844	330	331	$-0\frac{30}{100}$
	1 950	342	351	$-2\frac{56}{100}$
	1 565	432	422	$+2\frac{36}{100}$
201.5	1 776	471	465	$+1\frac{29}{100}$
	1 946	503	497	$+1\frac{20}{100}$
	1 605	876	855	$+2\frac{46}{100}$
320	1 798	958	936	$+2\frac{35}{100}$
	2 088	1 083	1 100	$-1\frac{54}{100}$

The examples of 255.5-foot span remain as before. It will be seen that the results are even better for Mr. Pegram's data than for my own. It is quite possible that Mr. Pegram's values are correct as published. But I thought it due to consistency to call attention to the discrepancies noted.

In a paper kindly sent to me some time ago by Professor Waddell is a list of carefully estimated weights of single track spans from 60 to 300 feet. The locomotive system used is very closely that which I have assumed in this paper. The values of $w_2 + w_3$ are given. We are able, therefore, at once to check our formula by Mr. Waddell's estimates. The first three spans are single intersection pony trusses, the rest down to 160 single intersection through, and the remainder double intersection through trusses. We have found, by applying our formula to a few cases, that Mr. Waddell's value of μ is about 6 540 for single and 5 830

for double intersection. Using these values of μ_1, taking w_1 from Table No 1, and $w_2 + w_3$, as given by Professor Waddell, we have the following results :

Span in feet, l.	Depth in feet, d.	Number of panels, N.	$w_1 + w_2 + w_3$.	w_0.	L.	w_4 by formula.	w_4, as estimated by Prof. Waddell.	Per cent. of difference.
60	8	5	2 181	50	16.613	193	214	$-9\frac{81}{100}$
65	8	5	2 130	50	14.499	212	231	$-8\frac{22}{100}$
70	9	5	2 090	51	12.887	217	224	$-3\frac{12}{100}$
S. I. 70	23	5	2 089	75	19.377	193	209	$-7\frac{65}{100}$
80	16	5	2 048	64	15.577	208	215	$-3\frac{25}{100}$
90	23	5	2 046	75	14.871	227	221	$+2\frac{26}{100}$
100	20	5	2 065	70	12.461	256	252	$+1\frac{58}{100}$
110	23	6	2 069	83	11.082	296	281	$+5\frac{33}{100}$
120	21	6	2 088	80	9.563	333	319	$+4\frac{38}{100}$
130	23	7	2 102	90	8.675	376	371	$+1\frac{34}{100}$
140	23	7	2 086	90	7.828	408	412	$-0\frac{72}{100}$
150	23	7	2 080	90	6.664	473	452	$+4\frac{64}{100}$
160	24	8	2 076	98	6.460	496	495	$+0\frac{22}{100}$
170	26	8	2 079	103	6.128	528	544	$-2\frac{94}{100}$
180	27	8	2 081	106	5.742	567	571	$-0\frac{86}{100}$
190	28	9	2 065	114	5.239	628	640	$-1\frac{88}{100}$
D. I. 180	30	8	2 281	114	5.892	604	569	$+6\frac{15}{100}$
190	32	9	2 065	125	5.578	603	631	$-4\frac{43}{100}$
200	33	9	2 068	128	5.242	646	664	$-2\frac{71}{100}$
210	34	10	2 068	136	4.904	701	725	$-3\frac{31}{100}$
220	36	10	2 068	143	4.705	740	755	$-1\frac{98}{100}$
230	38	10	2 069	150	4.522	780	782	$-0\frac{25}{100}$
240	39	11	2 073	158	4.254	843	847	$-0\frac{47}{100}$
250	41	11	2 079	165	4.101	889	900	$-1\frac{22}{100}$
260	42	11	2 082	170	3.916	942	946	$-0\frac{42}{100}$
270	43	12	2 087	178	3.703	1 016	1 010	$+0\frac{59}{100}$
280	44	12	2 095	182	3.549	1 075	1 055	$+1\frac{89}{100}$
290	45	13	2 098	190	3.367	1 157	1 131	$+2\frac{30}{100}$
300	46	13	2 104	194	3.237	1 221	1 188	$+2\frac{77}{100}$

As I presume Professor Waddell would not claim for his estimates absolute exactitude, the correspondence of the formula as that of estimate with estimate seems all that could be desired. The short pony spans are all greater than formula, as they should be, as there is no variation of cross-section in such cases, and hence an excess of material.

More tests are wanted, but I need hardly point out that they must be of well and uniformly designed spans, for the same or uniform specifications, and actual data must be used. For different designs and styles the value of μ may need to be changed. Thus in Professor Waddell's case we have two values, one for single and one for double intersection. But for same design and style, this value should be constant. This is equivalent to saying that if we take μ at 8 000 our co-efficient 3.6 includes really a "co-efficient of design." In the case of Mr. Pegram this co-efficient is unity. For Professor Waddell's cases it is 0.8175 and 0.72875. In any set of uniformly designed trusses it is simply necessary

to find by reversing the formula, the average value of μ and then using this average value, the formula can be trusted to reproduce the entire set and give the influence upon weight of any change in dimensions.

If no such uniform value of μ is found I think I am justified in concluding that either the spans are not uniformly designed to the same specifications or else that some of the data are incorrectly assumed.

Mr. Hughes' Spans.

This is, I think, the case with Mr. Hughes' cases, quoted in my previous paper.

Thus one of his examples is double intersection,

$$l = 200,\ d = 28,\ N = 13,\ w_1 + w_2 + w_3 = 1\,560,\ w_4 = 474.$$

This is almost identical in every respect with one of Mr. Pegram's cases, and yet the weight as designed by Mr. Pegram is $w_4 = 422$. Taking $\mu = 8\,000$ for Mr. Pegram, we must take in this case $\mu = 7\,116$ for Mr. Hughes. But this value of μ will give only one or two of Mr. Hughes' other examples with such accuracy as we desire. The value of μ in order to give his results accurately must fluctuate considerably as is shown by the following table. The values of w_1 are taken from Table No. 1, and $w_2 + w_3$ as given by Mr Hughes. The last two spans are double track.

l.	d.	N.	$w_1 + w_2\ w + w_3$	w_4 actual value.	μ from formula.
S. I. 50	6¼	4	1 895	157	6 320
" 61	8	4	1 810	139	8 539
" 75	21	5	1 666	184	5 508
" 84	21	6	1 677	201	6 714
" 105	20	7	1 679	206	9 150
" 120	22.5	8	1 678	246	8 960
" 135	22.5	9	1 663	267	10 052
D. I. 165	27	11	1 616	382	7 028
" 184	28	12	1 573	414	7 356
" 200	28	13	1 560	474	7 116
S. I. 142	27	7	3 331	423	9 046
" 150	28	7	3 268	409	10 246

It seems to me improbable that a rational formula which agrees so well with the results of two designers, for both of whom μ has a constant value, should suddenly prove inaccurate for a third. I must conclude as already stated, either that there is some error in the data, or that the spans have not been uniformly designed from uniform specifications, and hence to quote Mr. Pegram's remark upon these same spans, "are not sufficiently consistent to entitle them to use as a standard." Our formula, even in Mr. Hughes' case, would give good results, but not such close correspondence as we seek.

More tests of the formula under proper conditions are needed, and I trust I shall be favored with them, or with all the necessary data to enable me to make them myself.

Least Weight Depth.

If we differentiate II and equate the first derivative to zero, we find for the depth which gives w_4 a minimum

$$\frac{d}{l}=\frac{1}{N}\sqrt{\frac{\alpha\left[1+\frac{1}{5(N-1)}\right.}{\beta\left[1+\frac{4}{5(N-1}\right]+\frac{1.2\,\mu\,N}{(w_1+w_2+w_3)+A}}}$$

The values of α and β are to be taken from Table No. 4, and of A from Table No. 3. For standard specifications and the locomotive system we have adopted, we may take $(w_1+w_2+w_3)$ equal to 2 000, without noticeable error.

If we use this value of $w_1+w_2+w_3$, we can make at once the following tabulation, which will enable us to find directly the least weight depth for any length of span, for Warren single or double intersection.

TABLE No. 5.

Least Weight Depth, $d=Cl$. Values of C, Given in Table.

N.	Warren girder C.	Single intersection C.	Double intersection C.
4	0.2207	0.2510	0.2592
5	0.1978	0.2258	0.2436
6	0.1805	0.2018	0.2272
7	0.1669	0.1846	0.2122
8	0.1559	0.1708	0.1992
9	0.1467	0.1596	0.1875
10	0.1389	0.1502	0.1773
11	0.1348	0.1420	0.1684
12	0.1263	0.1350	0.1605
13	0.1211	0.1290	0.1534
14	0.1164	0.1235	0.1470
15	0.1123	0.1196	0.1413
16	0.1084	0.1142	0.1360
17	0.1049	0.1102	0.1312
18	0.1016	0.1065	0.1267
19	0.0986	0.1031	0.1226
20	0 0958	0.0999	0.1187

If, then, our formula is accurate and follows correctly change in depth, we can easily find at once the depth for minimum weight. From Formula I and Table No. 2, and formulas for w_3, we can then find the weight of iron per foot for any number of panels. By a few trials we can thus find the best number of panels corresponding to the least weight depth, or, with any desired depth, can find the best number of panels.

As an illustration, take a span of 104 feet, single intersection.

For $N=4$, we have from Table No. 5, $d=26.1$ feet. In this case $w_1=1\,564$, $w_2+w_3=406$, and Formula II gives us $w_4=207$ pounds. We have then from I total weight of iron $=826$ pounds per foot.

For $N = 5$, we should find in like manner $d = 23.5$ and weight 820, and for $N = 6$, $d = 21$ and weight 840 pounds per foot.

We see at once that the best number of panels is N = 5. Local circumstances and peculiarities of design may determine the depth. In such case we can easily find for the given depth the best number of panels. In Mr. Pegram's example of 104-foot span, the depth is taken at 24 feet, or very closely what our formula gives it. But according to our formula 5 panels instead of 6 would have been better.

The formula also shows that the weight varies but little for a moderate change of depth, other things remaining the same. The least weight depth is then of minor importance except as indicating in any case the amount of deviation. The panel length is, however, of more importance, and our formula completely confirms the results towards which best modern practice has been constantly tending, viz., long panels.

The preceding will suffice to indicate the use to which such a rational formula as that which we present may be put, provided it is sound and accurate. It serves not only as a means of estimating weight but has scientific interest as a guide in design. As such uses depend entirely upon its accuracy, and as the accuracy is at present the precise point to be established, we shall not further indicate the directions in which the formula should prove valuable. They are sufficiently obvious. Until the accuracy of the formula is first established and admitted it would be a waste of time to attempt to largely apply it to problems of design.

Empiric Formula.

As variations in depth do not largely affect the weight of truss, and the value of $w_2 + w_3$ varies but little around a constant value, it would seem possible to construct a purely empiric formula, which should contain the span as the only variable, and give at once with little calculation and with sufficient accuracy for an estimate, the entire shipping weight.

Such a formula was presented by Mr. Pegram at the annual Convention, June 10th, 1884, and published in the Transactions for February, 1886. His formula was:

$$\text{Total weight of iron in pounds} = \left(a + \frac{l}{c}\right) l \sqrt{l}$$

By giving to the constants a and c, a series of values for the various load systems specified, and changing them when the weight deviated too much from the actual, this formula showed a good correspondence with actual weights of bridges designed according to the specifications.

We would present as equally simple and easy of application, and as more accurate so far as can be judged by correspondence, the following formulas:

$$\text{Total weight of iron in pounds} = \frac{a + b\,l}{\frac{c}{l} - 1}$$

For Mr. Pegram's "Class M," for plate girders, $a = 124\,720$; $b = 3\,415$; $c = 585$, and for truss spans under 320, $a = 301\,900$; $b = 562$, $c = 518$.

For "Class C," for plate girders, $a = 393\,880$; $b = 14\,848$; $c = 2\,040$, and for truss spans under 320, $a = 303\,000$; $b = 1\,160$; $c = 578$.

For "Class T," for plate girders, $a = 228\,612$; $b = 7\,774$; $c = 1\,110$, and for truss spans under 320, $a = 293\,343$; $b = 1\,167$; $c = 549$.

We give in the following table the comparison of the results of these formulas with the actual values as given by Mr. Pegram, also our respective percentages of error. All spans up to 80 are deck-plate girders, 8 feet wide, ties on top chord. Above 80, through Pratt trusses up to 150, and Whipple over that length.

It would seem that for the range covered, the proposed empiric formulas are considerably more accurate than those of Mr. Pegram. The constants a, b and c are easily determined for any series of executed examples, provided they are all uniformly designed in accord with uniform specifications. The values given, of course, hold good only for Mr. Pegram's cases. But any designer can easily make out the constants for his own practice. For preliminary quick estimate of weight such formulas will always possess value. I attribute the accuracy of those just given, to the fact that they are identical in form with that required by the rational discussion. Judging alone from the correspondence in the cases just given, this accuracy is sufficient for a very close estimate. While, therefore, these formulas may be used within proper limitations as giving close and ready approximations, and admit of being varied for individual practice, according to the specifications and requirements and style of bridge, the more general and rational formulas which have been given, possess, if accurate, a scientific interest and value, and take account of style and proportions.

Span in feet.	CLASS M. Moguls and 1 820 pounds.				CLASS C. Consols and 2 240 pounds.				CLASS T. Typicals and 3 000 pounds.			
	Actual.	Formula.	Per cent. of Difference.	Per cent. Pegram.	Actual.	Formula.	Per cent. of Difference.	Per cent. Pegram.	Actual.	Formula.	Per cent. of Difference.	Per cent. Pegram.
20	6 841	6 832	$-0\frac{13}{100}$	$+0\frac{30}{100}$	6 841	6 840	$-0\frac{1}{100}$	$+0\frac{90}{100}$	7 047	7 041	$-0\frac{85}{100}$	$-1\frac{20}{100}$
30	12 523	12 280	$-1\frac{14}{100}$	$+1\frac{70}{100}$	12 732	12 541	$-1\frac{50}{100}$	$+1$	12 979	12 828	$-1\frac{16}{100}$	$+0\frac{50}{100}$
40	18 635	19 179	$+2\frac{91}{100}$	$+6\frac{20}{100}$	19 283	19 756	$+2\frac{45}{100}$	$+4$	19 807	20 133	$+1\frac{64}{100}$	$+3$
50	27 639	27 614	$-0\frac{8}{100}$	$+1\frac{30}{100}$	28 561	28 550	$-0\frac{3}{100}$	$-0\frac{40}{100}$	29 115	29 123	$+0\frac{2}{100}$	0
60	37 360	37 670	$+0\frac{83}{100}$	$-0\frac{40}{100}$	38 590	38 932	$+0\frac{88}{100}$	$-1\frac{20}{100}$	39 502	39 717	$+0\frac{54}{100}$	$-1\frac{70}{100}$
70	48 739	49 445	$+1\frac{44}{100}$	$-2\frac{80}{100}$	49 983	50 962	$+1\frac{95}{100}$	$-3\frac{10}{100}$	51 033	52 040	$+1\frac{97}{100}$	$-2\frac{50}{100}$
80	63 061	63 036	$-0\frac{4}{100}$	$-7\frac{30}{100}$	64 561	64 560	-0	-7	66 061	66 067	$+0$	$-6\frac{50}{100}$
104	90 555	90 540	$-0\frac{1}{100}$	-2	93 050	92 964	$-0\frac{9}{100}$	$-1\frac{20}{100}$	96 957	96 940	$-0\frac{1}{100}$	$-1\frac{70}{100}$
130	124 871	125 656	$+0\frac{62}{100}$	$+2$	129 688	131 688	$+1\frac{54}{100}$	$+2$	137 285	138 086	$+0\frac{58}{100}$	$+1\frac{10}{100}$
150	154 427	157 439	$+1\frac{94}{100}$	$+4$	163 302	167 192	$+2\frac{38}{100}$	$+3$	173 849	176 087	$+1\frac{28}{100}$	$+2$
175	204 486	204 209	$-0\frac{13}{100}$	$+1\frac{60}{100}$	220 006	219 808	$-0\frac{90}{100}$	$-0\frac{70}{100}$	232 637	232 834	$+0\frac{8}{100}$	$-0\frac{50}{100}$
201.5	267 410	264 422	$-1\frac{11}{100}$	$-1\frac{80}{100}$	286 055	286 584	$+0\frac{18}{100}$	-3	304 459	306 550	$+0\frac{68}{100}$	$-2\frac{60}{100}$
255	441 065	431 823	$-2\frac{7}{100}$	....	468 884	472 985	$+0\frac{87}{100}$	$+4\frac{70}{100}$	508 415	512 513	$+0\frac{80}{100}$	$-1\frac{20}{100}$
320	778 552	778 255	$-0\frac{3}{100}$	....	837 387	836 476	$-0\frac{18}{100}$	$+0\frac{30}{100}$	946 031	932 563	$-1\frac{43}{100}$	$+2\frac{50}{100}$

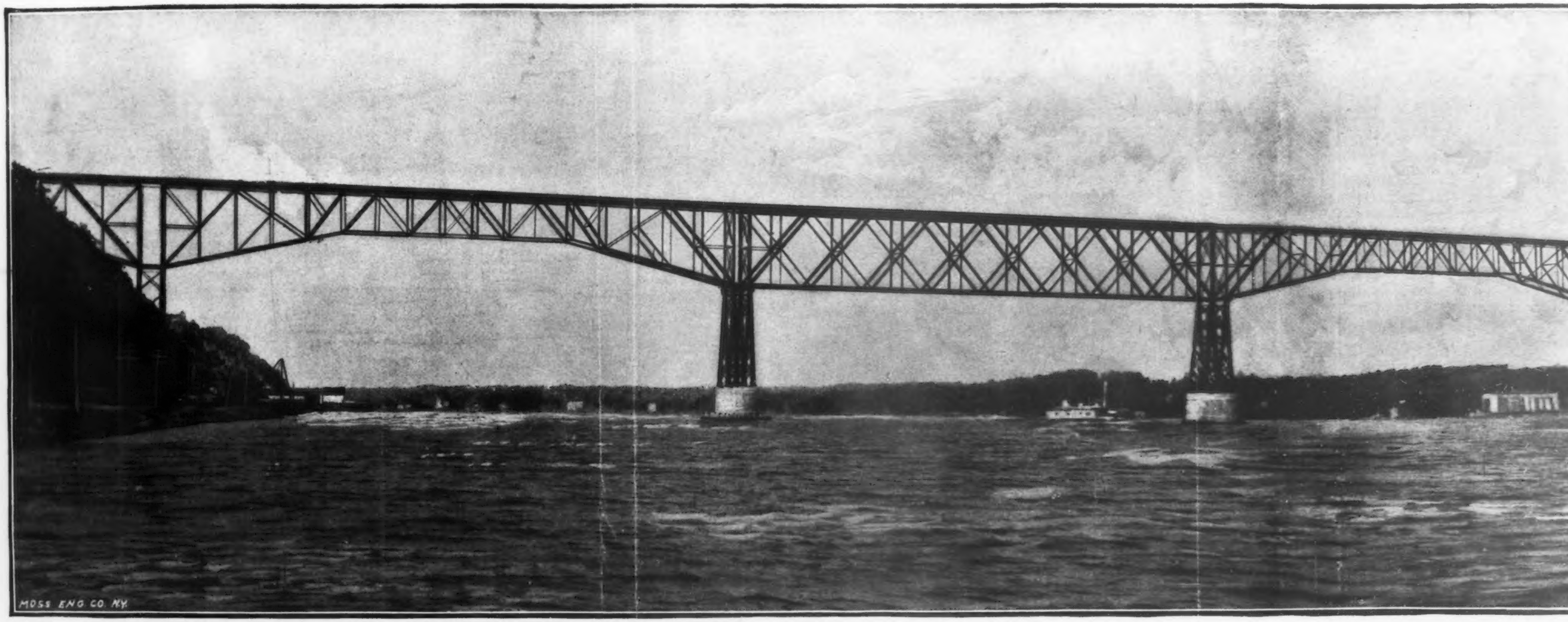


PLATE XL.

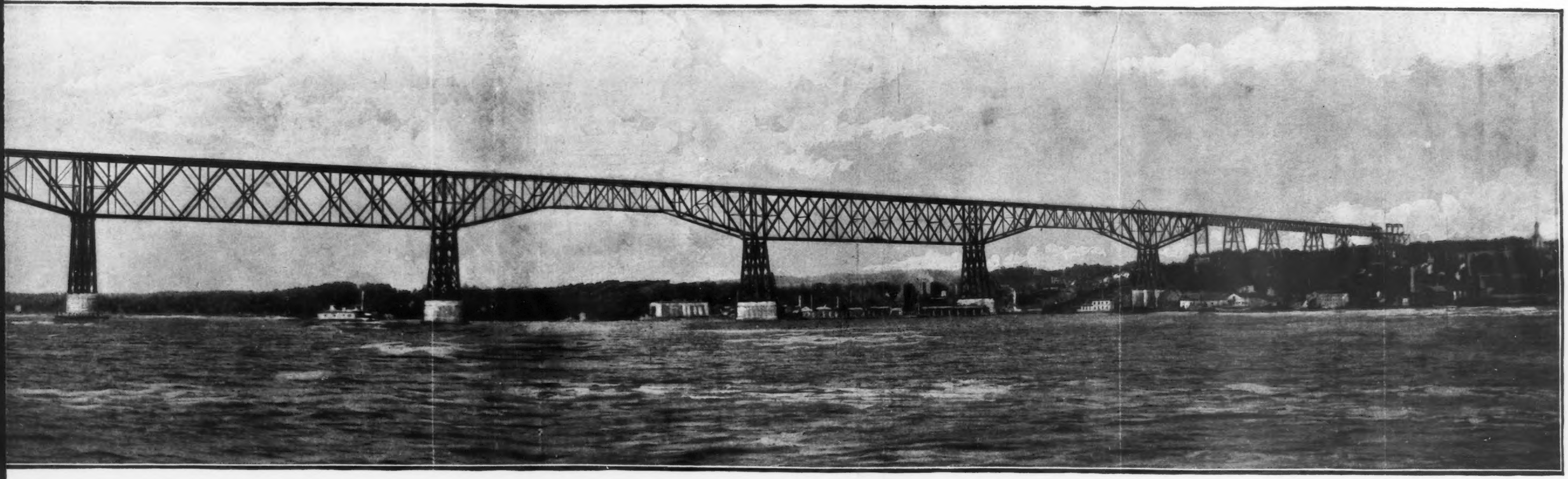


PLATE XL.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

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(Vol. XVIII.—June, 1888.)

THE CONSTRUCTION OF THE POUGHKEEPSIE BRIDGE.*

By JOHN F. O'ROURKE, M. Am. Soc. C. E.

Some years ago, when the scheme of bridging the Hudson River at Anthony's Nose was a subject of frequent discussion, it was suggested by the Poughkeepsie newspapers that the physical, geographical and commercial advantages of their city made it the best point at which to locate the proposed crossing. The idea grew among the citizens, and through their efforts, the Legislature of 1871 was induced to grant a charter for its construction. In it was a clause prohibiting piers in the river.

During 1871 the promoters had surveys made and the engineering features examined. Eminent authorities were consulted as to the possibility of spanning the 2 600 feet between the shores with a suspension bridge. Among others, Captain Eads declared the span too long for practicability and recommended four piers in the river, not less than 500 feet apart in the clear. These facts were presented to the Legislature of 1872, and the charter was amended so as to permit of their construction.

In 1873, the Pennsylvania Railroad, represented by J. Edgar Thomson and A. D. Dennis, subscribed $1 100 000; the capital stock being

*The substance of this paper was presented at the Convention of 1887, but was withdrawn for revision by the author.

$2 000 000. The control passed into their hands and the immediate building of the bridge seemed assured.

The charter required that the work should be begun before January 1st, 1874, so a part of the foundation of a pier was built on Reynolds' Hill, near the site of the present east anchorage pier, and on December 17th, 1873, in the presence of a distinguished company, the corner-stone was laid with elaborate Masonic ceremonies and general rejoicing. On February 22d, 1887, this stone, with its contents untouched, was removed, and placed without ceremony or addition, in the southwest corner of the anchorage pier adjoining.

The financial world was at that time suffering depression from the panic, and money was hard to get for anything, however promising. During the following winter Mr. Thomson died, and the Pennsylvania Railroad voted that no new work should be undertaken without the consent of the stockholders. This was not to be had, so the directors could do nothing, and the project rested until 1876, when a contract was entered into with the American Bridge Company of Chicago, and the actual work of building begun.

The first river pier from the west shore was built by them to a height of 20 feet above water; the crib of the next pier sunk through 55 feet of water and 40 feet of river bed, with its top 1 foot above high water; and a third crib built, 36 courses high, but not placed, when the company suspended work in 1878.

From then, nothing was done but to keep the charter alive by extensions of time, until 1886, when a number of gentlemen, forming the Manhattan Bridge Building Company, took up the project. They acquired the rights of the American Bridge Company's assignees, and made a contract with the Union Bridge Company for the entire structure complete.

In September, 1886, operations were begun. A line, practically the same as the old one, was adopted, surveys and borings were made, and a profile obtained, from which the location of the piers was determined.

The former design was for rectangular trusses of equal lengths. The location of the second span of the new design was fixed by the existing pier; and its length of 525 feet by the 500 feet of clear span and the 25 feet of masonry required. The same length of span to the west, would place Pier 1 in the West Shore Railway tracks. A pier on the

east slope of the railway would be inadmissible. The most available site was on the bluffs west of the highway, and this was selected. By this time a cantilever design had been decided on, and the 530 feet center to center of end pins necessary to reach these bluffs was made common for all three cantilever spans. This clear span, with the half-widths of towers added, made the spans next the shores 548 feet and the center span 546 feet, which with the two connecting ones, of 525 feet, center to center of piers, located Pier 6 in the face of the rock on the east shore. The total length between anchorages was thus established at 3 093 feet 9 inches, the total length, including viaducts, 6 767 feet 3 inches.

The charter fixed the bottom of trusses at 130 feet above high water; so the height of trusses proportionate to the spans required the adoption of 212 feet above high water as the grade of base of rail. The west approach has a rise of 66 feet per mile westward out of the valley of the Hudson, and the same grade was adopted on the viaducts, since it did not further limit the size of trains.

After the adoption of the general design, the next thing was an accurate location. The West Shore road-bed is level and advantageous for base lines. Two were laid out there, each about half a mile long, intersecting the center line at a common point. The triangles had a point on center line on the east shore for a common vertex, and were nearly isosceles. The bases were measured with a common 100-foot steel ribbon with brass handles, and without a spring balance or leveling attachment. Chaining in the sunshine was avoided, so correction for temperature was exactly determined. Plugs were driven flush with the ground every hundred feet, line taken, and then, with the chain suspended, and under strong tension, a distance point was decided on after a few trials. On level ground, like the West Shore road-bed, errors of over a half inch to the mile would be carelessness.

The angles were measured with a Heller & Brightly tunnel transit. Twenty repetitions were made at each station, a reading was taken every time and the difference from the preceding obtained, to guard against errors caused by the slipping of the plates. One-twentieth of the twentieth reading gave the angle to the fraction of a second. A third triangle was measured from the south base line with a vertex on center line 82.07 feet west of that used before.

The angles of the north triangle had a deficiency of 7 seconds, those of the south triangle balanced exactly, and those of the third had an excess of 1½ seconds.

The distances as calculated were respectively 2608.66 feet, 2608.78 feet, and 2608.68 feet, average 2608.71 feet. 2608.82 feet was the distance subsequently obtained by measuring across on the ice. Some more recent triangulation of two points on the eastern shore, invisible from each other, for use in locating cribs 4 and 5, incidentally gave this distance as 2608.60 feet, although great accuracy was not intended. The triangulation to crib 3, pier 2, and upon the west bluffs, was equally satisfactory, several independent triangles differing from each other only minutely.

These triangles are all shown on the accompanying map, Plate LVI.

Work proper was begun October 8th, 1886, excavating for foundations of Pier 1; that for Pier 6, October 20th; east and west anchorages, November 10th.

Masonry of Pier 6 was begun December 6th, 1886, and finished February 17th, 1887; Pier 1, January 7th, 1887, finished March 17th; east anchorage, February 8th, finished March 23d, and west anchorage, January 26th, finished May 9th.

All shore work is first-class bridge masonry, the hearting being generally concrete. Built in each anchorage pier are four iron girders, underneath which are cross-girders, connected with eye-bars to the pedestals of the shore arms. The piers are heavier than is necessary to resist any possible effort of the cantilever spans to lift them.

The borings show the river bottom to be composed, for more than 100 feet below high water, of various combinations of mud, clay and fine sand, too soft for building upon. Underlying this pasty stuff is a very firm and hard stratum of rather coarse sand, beneath which is gravel, and about 140 feet down, solid rock extending from shore to shore.

The general design of a pier is a crib and grillage, extending from the gravel to 10 feet below high water, on this is the masonry to 30 feet above high water, upon which is a steel tower 100 feet high to pedestals of trusses. Plates XLIV, XLVI, XLVII and L.

The cribs are practically alike, so No. 5 will be described. The base is 60 x 100 feet, and the height 104 feet. It is built of 12 x 12-inch white hemlock, except the bottom course, which is of white oak. The lower part or shoe is composed of five prisms of solid timber, 20 feet high, the cross-sections of which are triangular with the bases on top, the vertices lying in the five cutting edges which divide the bottom into two rectangles each, 30 x 100. The top surface of this shoe is 10

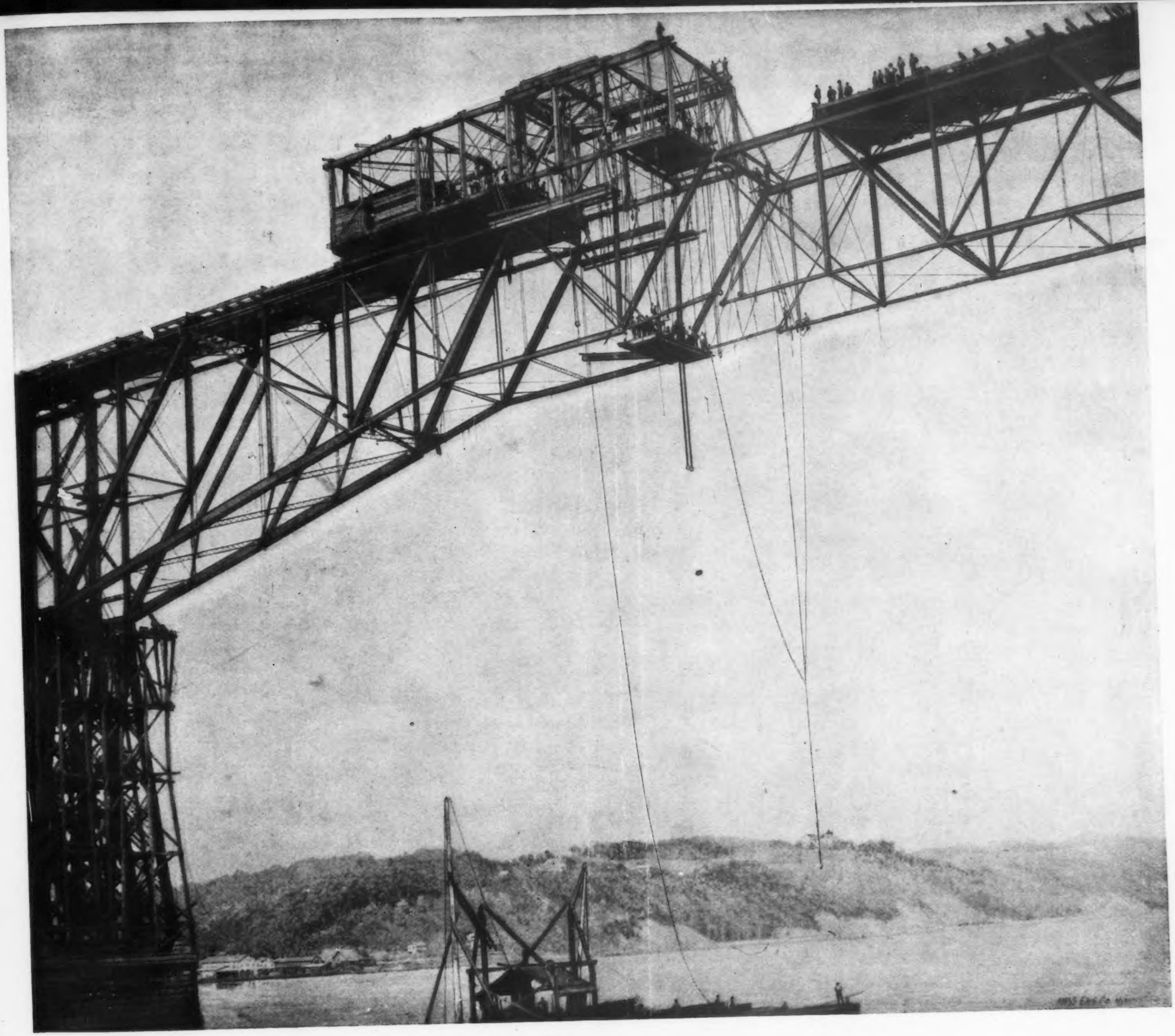


PLATE XLI.

feet wide at the sides, 9 feet at the ends and 16 feet in the middle. These prisms inclose two truncated pyramidal spaces, the tops of which are 12 x 82 feet. These spaces are divided into fourteen smaller ones, by six bulkheads of timber, 2 feet thick, that are built into the solid cribbing from the cutting edges up. These subdivisions are called the dredging chambers, and are 10 x 12 feet on top. Upon the shoe rest the weighting pockets. The walls of these are of 12 x 12 timber, 2 feet thick, and built along on the edges of the top surface of the shoe, and on the bulkheads of the dredging chambers. They, in every instance, intersect each other and extend from out to out of the crib. The building is done without halving, by laying all the longitudinal timbers of one course across the intersections, the transverse ones being cut the length of the spaces between. The next course is laid with the transverse timbers running across the intersection, while the others are fillers.

The drift bolts used are 1 inch round by 30 inches long, 450 to each course. The number was determined by learning from experiment that a 1-inch bolt driven into a $\frac{15}{16}$ hole adhered about 422 pounds per linear inch. It was assumed that the buoyancy of all the parts above the solid cribbing could act at once on all the bolts through that joint. The number of bolts giving a safe excess of resistance was made uniform for all the courses.

The openings over the dredging chambers are the passages through which the dredge bucket descends. They are called the dredging pockets.

Cribs may be built for some height on ways and launched, or they may be built on the ice about three courses high and cut in. Building is continued alongside a wharf until the draft is nearly the depth of the river, when it is taken to the pier site, anchored, set, the weighting pockets loaded with gravel to keep the top at a convenient distance above the water, and built upon until it is embedded in the mud. Weighting, building and dredging are then carried on with more or less continuity until the crib has reached its final resting place.

The borings give an approximate idea of how far a crib should go to reach a reliable bottom, sufficiently close for general plans, but the dredge bucket is better, and a close attention to its working and contents gives a true idea of the character of the strata.

Twenty feet below high water was adopted as the depth to which the

top of the crib was to be sunk, so it follows that it must be finished off, while it has yet about twenty feet to go. When dredging begins there is no side friction. The material is soft, and the crib follows the dredging easily and evenly. When the crib has penetrated perhaps 20 feet into the bottom, the material is more compact and side friction has become considerable. It no longer moves steadily, but by intermittent, though gradual descents. Toward the end the crib hangs until the bottom is undermined, when it drops sometimes as much as 10 feet at once, its motion being like that of a weight set free on the surface of a thick, soft cushion; and it comes to rest without jar. The dredge can only dig well holes, whose areas are about one-fourth that of the bottom of the crib, and it must depend upon the material falling in from the sides for the greater part of the excavation. These holes are often 30 feet below the cutting edge when in hard material, which is not sand, so a timely knowledge is obtained of how far down the crib should go, and to what height it should be built. The top course timbers floor over the entire surface, except the dredging pockets, after the weighting pockets have been filled flush with the course below. Upright timbers, about thirty feet long, are then fastened to the inner sides of alternate pockets to indicate to the dredge runner their position when under water, and clusters of piles are driven around the crib for the dredge to lie against when the former is below the surface of the water. Toward the end the dredging must be done with great care, and only a few bucketfuls removed from each pocket as the dredge goes round and round the crib. It was found that when out of level in hard material a crib righted itself if the material was left highest on the high side, as the momentum in falling is greatest there. In soft material, the crib settling slowly along with the dredging, that keeping the bottom low under the high part, remedied the difficulty. When the crib has made its final descent all the loose material is dredged out, and filling the dredging chambers and pockets with concrete is begun.

The concrete is lowered into place in boxes carrying a cubic yard; a tag line attached to a trip unlatches them at the proper time, and it is deposited without being scoured. These boxes are hung over the ends of scows, from which project the mixers. The latter are simply rectangular boxes of plate iron, provided near each end with cast-iron collars, each grooved to retain itself on a couple of wheels, and are turned by means of a small engine which forms part of the machine. A jet of

water inside and a curved pipe to the level of the mixing deck for feeding, completes an effective machine. The mixer is set at an inclination of about 4 degrees, so that the concrete may pass along as it is rapidly turned over and over. On the deck of the scow gravel is mixed dry with 1¼ barrels per cubic yard of cement, whence it is wheeled in barrows or cast into the feed pipe. By this method 400 cubic yards are placed by two outfits in ten hours. The accompanying drawing, Plate XLVII, shows the operation at Crib 3. When the crib is filled with concrete to within 2 feet of the top of pockets, the remaining part is filled with broken stone and leveled by divers.

The floating caisson is then towed to place over the crib, fastened with four anchors, and the masonry begun. When the caisson is just afloat at high water only, it is accurately set, and masonry and loose stone enough to hold it down put in before the next high tide. Then an exact location of the pier is made, using a suspended piano wire to lay out the span. From this point on the work is ordinary building until the masonry is completed and ready for the steel tower. The ashlar is in courses varying in thickness from 2 feet 9 inches to 1 foot 10 inches. The use of concrete backing, with large stones imbedded in it, enables about 125 cubic yards per day to be laid, and gives a pier which is practically a monolith.

Domestic cement is used in the concrete for the cribs, Portland cement in the masonry. The mortar is mixed of one part cement and two parts sand, and the concrete backing contains 1½ barrels per cubic yard. All cement is rigidly tested in the manner recommended by the American Society of Civil Engineers. Ten per cent. of the barrels are sampled.

The testing machine used was designed by the writer for this work. Its capacity, which can be readily increased, is 1 000 pounds, and it is simple, compact and reliable. From the drawing attached, Plate XLIII, it can be easily duplicated by an ordinary mechanic.

It is worthy of note that the best results were obtained when the opening between the jaws of the clips had been increased to 1¼ inches. With a smaller opening the briquettes broke where held, instead of at the smallest section. Even with the enlarged opening the older briquettes often break at the clips.

Pier 2, as before stated, was built to 20 feet above high water by the American Bridge Company. It was 22 by 68 feet, and was to have been

carried to top of bridge. The steel towers designed by the Union Bridge Company require piers 25 by 87 feet, and consequently the old masonry was too small. (See Plate XLVIII.)

The crib of this pier is like the one described, differing only in that it is 10 feet narrower, that all the pockets are filled with concrete, and that it has iron cutting edges. It was begun in August, 1876, and launched in March, 1877.

During that season it was brought to place and completed. Its upper eight courses were calked, and a coffer-dam was built upon it. This was begun while the top was yet out of the water, and continued as the crib settled.

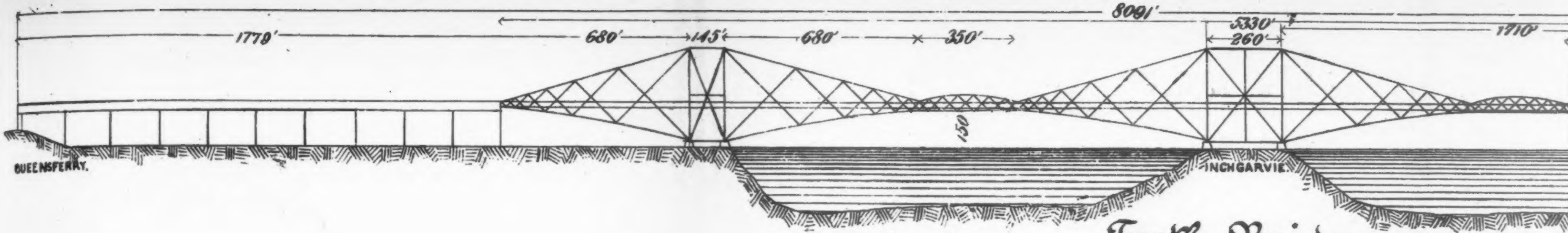
When in place, the cutting edges were 100 feet below high water, and the top 30 feet below same. After the dredging chambers and pockets were filled with concrete, the coffer-dam was pumped out. The upward pressure of a 38-foot head proved greater than the united resistance of drift bolts, concrete and weight of everything above bottom line of calking. The whole mass above that line lifted at the north end, hinging at the south end. As the water entered it settled back, but the bolts and débris prevented its closing, and left a space of about 8 inches at the north, which runs out to nothing before reaching the south.

A bearing below this rupture was obtained by sinking a pneumatic caisson 8 feet through the crib. It is 16 feet high, and the top is 22 feet below high water. Upon this rests the masonry.

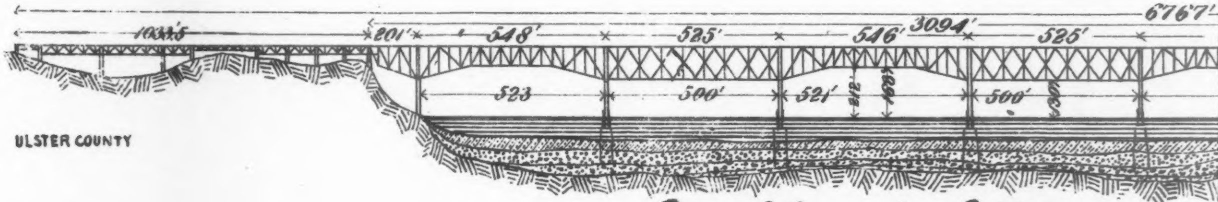
To remove this masonry from the top of the caisson, or to in some way avoid doing so, and yet construct a good homogeneous base for the larger pier, was the problem.

The crib is 50 by 100 feet, the pneumatic caisson 29 by 74 feet and 8 feet higher, so it was decided to sink a coffer-dam 43 by 96 feet 6 inches down on the top of the crib, fill up the inside with concrete to a point a little below that from which it would be necessary to remove the stone, pump out the coffer-dam, take down the old masonry, tie together securely the coffer-dam, concrete and old pier, and cover the whole with more concrete, upon which the new masonry was to rest.

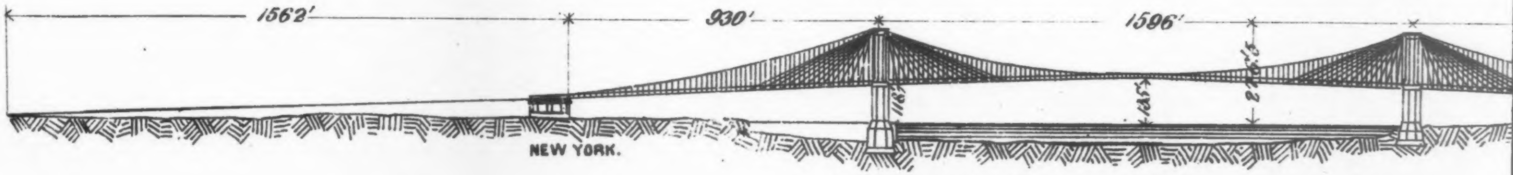
The building of this coffer-dam was begun January 20th, 1887. It is substantially two thicknesses of 12 x 12 timbers laid horizontally and separated by vertical timbers of the same size. The horizontal timbers were fastened through each post with $\frac{3}{4}$-inch round by 36-inch screw-bolts, the ends being held down by $\frac{3}{4}$-inch square by 24-inch drifts.



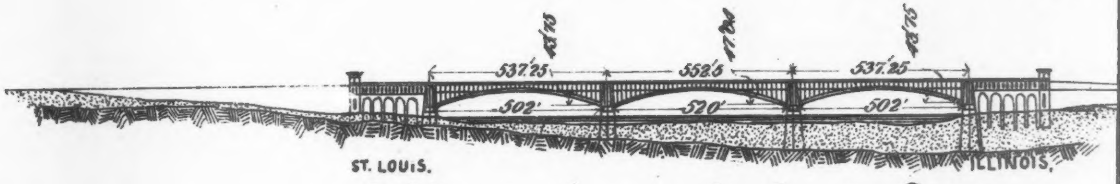
Forth Bridge.



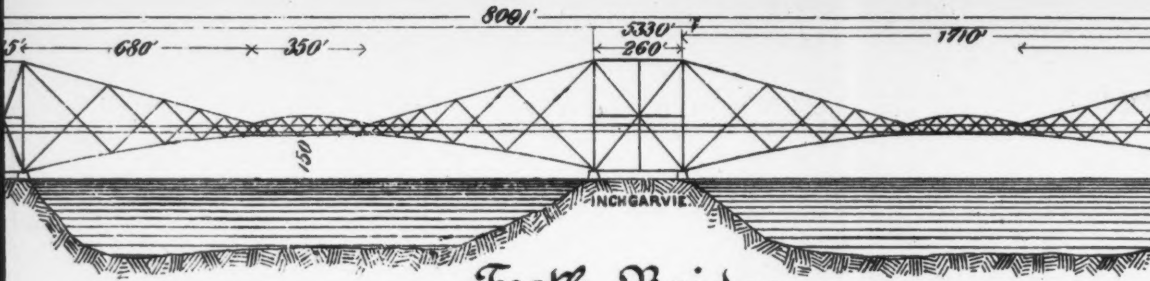
Poughkeepsie Bridge.



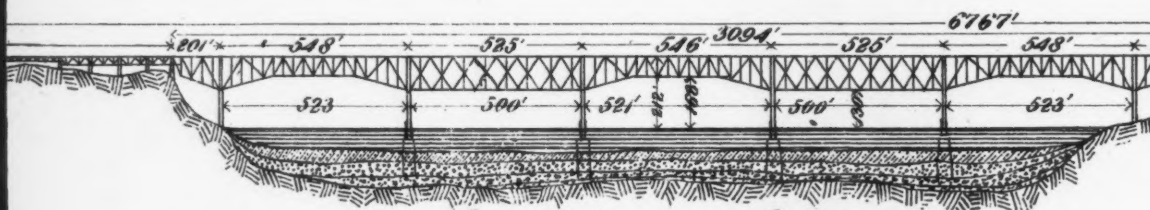
New York and Brooklyn Bridge.



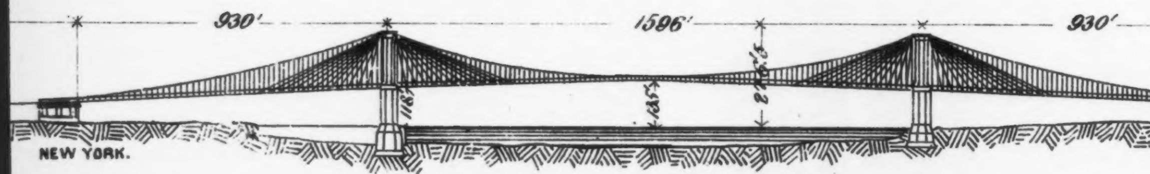
Illinois and St. Louis Bridge.



Forth Bridge.



Poughkeepsie Bridge.



New York and Brooklyn Bridge.

Illinois and St. Louis Bridge.

PLATE XLII
TRANS. AM. SOC. CIV. ENGNS.
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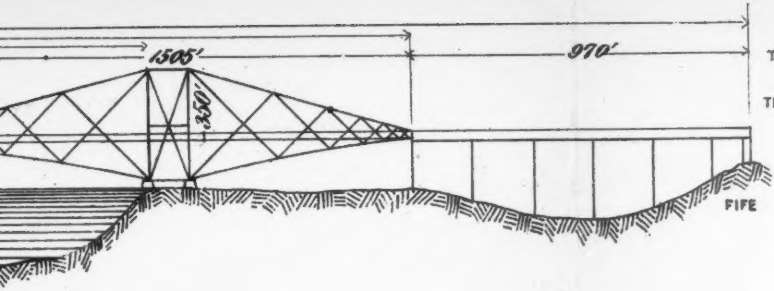
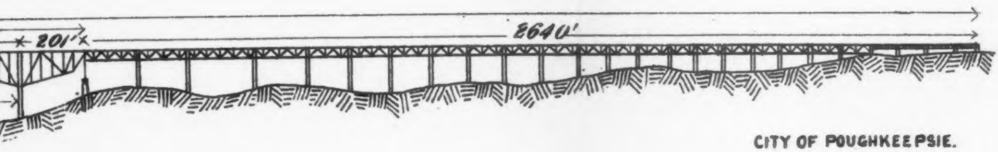
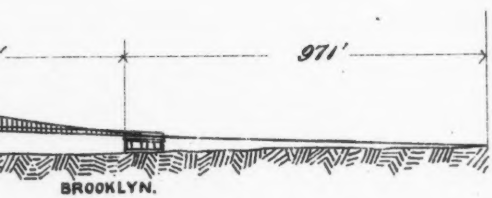
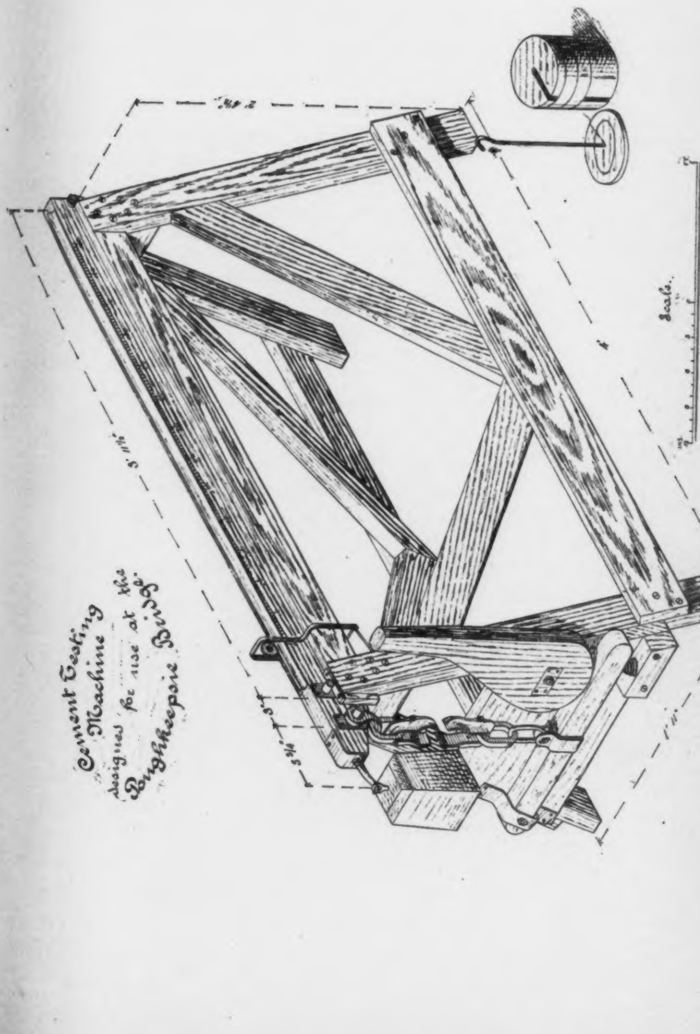
O'ROURKE ON
THE POUGHKEEPSIE BRIDGE.









Capacity, 1000 lbs.

PLATE XLIII.

The bottom course had 12 x 12 fillers, making it solid, so that the spaces between the posts were pockets. These were filled 16 feet high with concrete, and the remainder with clay.

Building it was begun on the ice, and after three courses were laid it was cut in. About one-third of it floated above the water, and every course was calked on the outside before immersion. When some twelve courses were laid, concreting the pockets was begun and carried on at a rate to keep the top conveniently high above the water. This concrete was lowered in boxes 10 x 16 x 36 inches, and discharged through the bottom by means of a latch and tag line. When the filling changed from concrete to clay, the latter was shoveled in and rammed as well as could be done. It may be said here that it was a mistake to use clay, for it could not be rammed solid, and was constantly escaping and giving trouble. The coffer-dam was finished March 1st, 1887. In a few days it was brought to a good bearing on the crib, and all the inequalities under it were leveled by the divers with concrete in bags. Some of the concreting inside the dam was done while the ice would bear, after which, until the latter went out, work was suspended. When the river was again clear, concreting was resumed and carried up to 12 feet below high water Then the dam was pumped out and the old masonry removed to 9.5 feet below the same. Across the top of this were laid $1\frac{1}{2}$ x 2-inch iron bars; attached to each pair were semicircles 5 feet in diameter of the same size iron, having $\frac{1}{2}$ x 8-inch plates riveted on their inside. These hung over the masonry to a foot or two below, and were attached to the coffer-dam with stirrups of 2 x $\frac{3}{4}$-inch iron. The bolts of the dam have the nuts inside, so the ends of the stirrups simply took the place of the washers. Four similar bars run lengthwise through the pier, and are fitted to the ends of the dam in the same way. Everything was then wedged to a bearing, the water was admitted, and the whole covered with concrete to a foot above the old masonry. After a few days had been given the concrete to harden, the dam was again pumped out, and the new masonry begun. All the concrete used was composed of two parts of Alsen's Portland cement, three parts sand and six parts screened gravel. It was all deposited in water, and samples brought up by the divers as the work progressed, showed it to be as good as if it had been deposited in air.

The crib of Pier 3, as before stated, was 96 feet high, and the cutting edge down to 95 feet below high water when the Union Bridge Company

took the work. Some rotten timbers in the three top courses were replaced by sound ones, and four feet of solid cribbing was added, to bring it to a height of 100 feet, after which the dredging was begun. This went very slowly at first, and the crib did not settle until sand was reached, the buckets making only wells down through the clay. The sand, however, flowed into the holes, and the clay being undermined, the crib settled six feet at one end and four feet at the other. This was four weeks after dredging began. For the next two weeks it went down with comparative uniformity, when it again hung, the cutting edge being at elevation 118 feet. This is the level of hard sand, and as it was desired to get it into the latter a few feet, the dredging was continued. On May 21st it dropped between 7 and 8 feet, canting, at the same time, a couple of feet to the west. It had been leaning in that direction from the first, and the result of this last move was to make the center of crib 5 feet east of center of span, 1 foot higher on the east than on the west, and 1 foot higher on the north than on the south. The crib being 60 feet wide and masonry 25 feet, there is yet 12½ feet of crib outside the masonry on the east. As the resultant of pressures and center of bottom of crib practically agree, we felt no doubt of its perfect stability, yet for convenience in construction a week or more was spent dredging to the rock, and outside, on the east, to bring it level, but the crib remained immovable. The masonry is laid on a bed of Portland cement concrete, the top of which is level when the caisson rests on the crib.

Crib 4 had been moored nine years at the Whale Dock, Poughkeepsie. It was towed from there on Thanksgiving Day, 1886, to the south end of Crib 3. Five feet of rotten timber were removed, and then building upon it was begun with the thirty-second course and continued to the fifty-sixth, when it was ready for anchoring in place. This is a difficult operation under the most favorable circumstances, with the dead load of more than 5 000 tons. It is an unwieldy bulk, having a large vertical outside area submerged, and several thousand square feet additional of walls inside against which the current from underneath would act. It was drawing 52 feet of water.

The first preparation was to place anchors in the river and to leave the ends of the cable accessible. These anchors, as shown by the drawing, Plate LVII, are simply cribs, measuring on the inside 6 x 6 x 6 feet, and contain 8 cubic yards of broken stone. They are composed of 3 x 8-inch hemlock planks, 10 feet long, piled on each other so as to leave

the above inside dimensions. Through the corners are eyebolts of 1¼-inch round iron, which serve the double purpose of binding the whole together and carrying the rope slings to which the cables are attached. There are also light screw-bolts through the middle of the sides to carry a cross-piece for reinforcing the bottom. When ready for filling these cribs are placed on greased ways on a scow, in the manner shown, usually one on each corner, and fastened with rope to the deck. They are filled with stone, the tops are put in, and, if not done before, slings of 7-inch rope are spliced through each pair of eyes and wrapped with sail cloth where liable to chafe. The slings are then gathered in iron thimbles, round which the cables are bent, and fastened with three clips each. They are then ready for launching.

The mode of placing is to tow the scow out to a range line marked on the nearest shore and shift about on that until signaled by a transit-man, when the rope attaching the anchor to the deck is cut, allowing it to slide overboard. The cable is slowly paid out until the end is brought to the pier site and buoyed, or made fast to a scow, in either case marked with a tag.

In the beginning three up-stream and three down-stream anchors, with two lighter ones for each side, were thought sufficient. At the first attempt to set Crib 4 there was delay in unmooring, and the spring freshet just setting in made the ebb tide early, so it had commenced running when we got away. Before proceeding far it grew too strong for the steamer, and the latter dropped back to Crib 3. Preparations were made to draw the crib up again where it had been. Two more steamers were attached, a cable to Crib 3 was put on a capstan and a line was connected to the cable of one of the anchors. The tide grew too strong, however, the new 9-inch towing hawser attached to the largest steamer broke, then the capstan was dismantled as the crib commenced to drift, and soon it was going down the river dragging three steamboats and one anchor with perfect ease. The crib was drawing within a few feet of the depth of the river in the deepest place and grounded once or twice, but the tendency in such cases is to follow the trend of the stream. It had gone three miles when the tide turned, after which it was brought back at midnight to Crib 3 without accident. This was, perhaps, the most difficult feat accomplished in connection with the work. A full and very bright moon, and an entire absence of wind alone rendered it possible.

This experience and the fact that the freshet was hourly growing stronger gave a different idea of the force to be contended with. The number of up-stream and of down-stream anchors was increased to five, the ends of the cables were this time placed on a scow at the pier site, and on the afternoon of April 11th the crib was again towed out. The five up-stream anchors were fastened on before the tide grew strong, but when it did, the crib commenced dragging them slowly. Then surging set in, and soon the northeast cable parted, after that the northwest one, and then it dragged the other three anchors very readily. There was just time to fasten the five down-stream anchors when the crib brought up on them and was held. So it stood for three days, until the same number of anchors was again ready. Two of the up-stream anchors were behind clusters of piles. This time the crib was securely moored in place after several hours' hard work. There was some motion when the cables were drawn up tense. It was expected to stop this by grounding, but the bottom scoured as fast as the crib settled. On the 19th it commenced surging badly and dragging the anchors, and on the morning of the 20th parted the cables of the anchors behind the piles, dragging the others 300 feet down stream. On the next ebb tide several more cables were parted, the crib commenced to drift slowly down stream, and it seemed as if it would get away. The freshet at this time was so great that the tide did not run up stream in rising, and when the ebb was at its strongest, seemed to run in surges, one moment backing off from the cribs smooth and gentle, and the next breaking at one or both corners in angry potholes. The crib surged and dragged and plunged with exceeding force, and it seemed as if nothing could hold it. New anchors were being constantly attached, scow loads of which were following it up. In launching them allowance had to be made for the distance the crib would drift while the ends of the cables were being brought aboard and fastened, so that they would just reach and give the most favorable lead. Soon there were ten on which held. Two steamers kept the crib from turning around, and so things stood until the 22d. At this time half the anchors were parted, and she had to be brought up before the preparations were entirely complete, or else go out with the next tide. The crib was now drawing 56 feet, and on the way back to place had to crush several of the lost anchors. We were now very expert in handling cables, and soon had six anchors on her bow, as the up-stream end might be called. A cluster of piles was in front of each up-stream

anchor. There were also five side anchors, two cables attached to Crib 3, and three anchors down-stream, when we left late that night, but in spite of all, the crib surged badly and two steamboats were kept pulling to steady it. The next morning two additional bow anchors and four more side anchors were put in. When they were all tightened up to perhaps 10 tons' tension each, the crib was immovable. Then it was felt that she had made her last trip. Two days more were occupied in setting her in position, clamping the cables permanently on horns outside, equalizing the tension, and removing the tackle.

It may be asked why we so underrated the forces to be controlled. At first, there was nothing to guide us but the experience of the builders of the first two cribs, and we used precisely what succeeded with them. Modifications were made as the necessity seemed to grow, and success was finally achieved when the working strength of the cables was nearly 1 000 tons. Their aggregate length was about three miles. Such a freshet in the Hudson is unexampled, and the work was done when it was at its worst.

The current, as measured, was about 2½ miles per hour, on an average, but there were times when it seemed more than twice that. There is no way of obtaining the exact pressure due to this cause under the circumstances, but one of the effects observed may give a good approximation to the stresses in the cables. When the tide was strongest, the end of the crib opposed sank two feet deeper than in still water, the other end rising about 6 inches. This is probably due to the vertical components of the stresses in the up-stream cables. The displacement was 2 000 cubic feet, or about 60 tons, of which, perhaps, 40 tons go to that end. Length of cable is to depth of water as eight to one, which would give, approximately, 320 tons as total stress in cables at north end, of which, perhaps, 240 tons went to the eight up-stream cables; the remainder to those side ones which were a little up-stream. So it would appear the cables were worked up to their limit, which was shown also by the strands drawing in places.

The anchors of Pier 5 were placed eight up-stream, six down-stream, and eight at the sides, as shown by drawing, Plate LVIII. The use of piles in front of them was abandoned, since the anchors never drew up to the clusters, except when the crib dragged from surging. They were placed in groups, so as to draw in nearly the same line. For putting the cables under tension, an engine having six spools was used with great success,

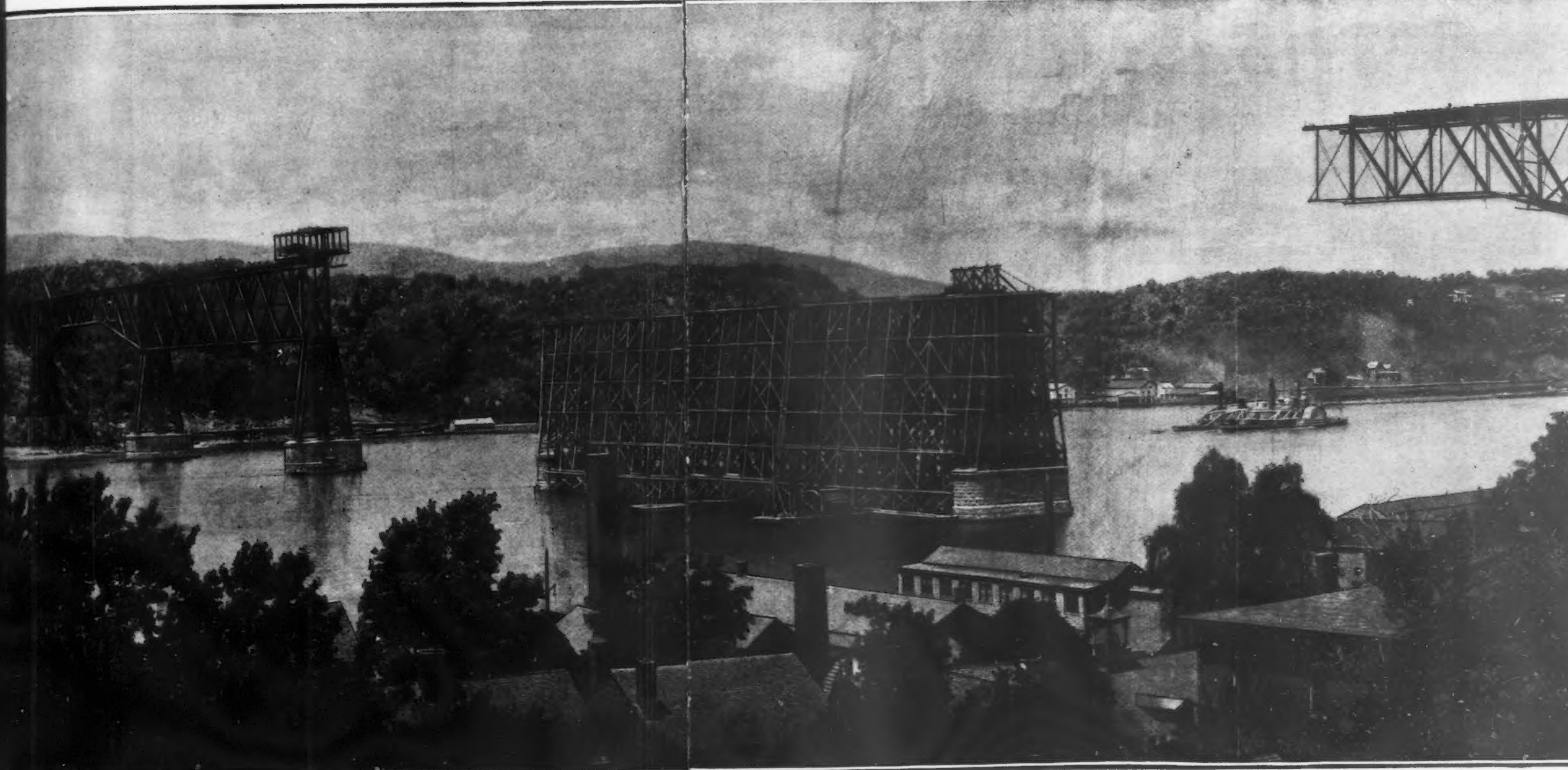


PLATE XLV.

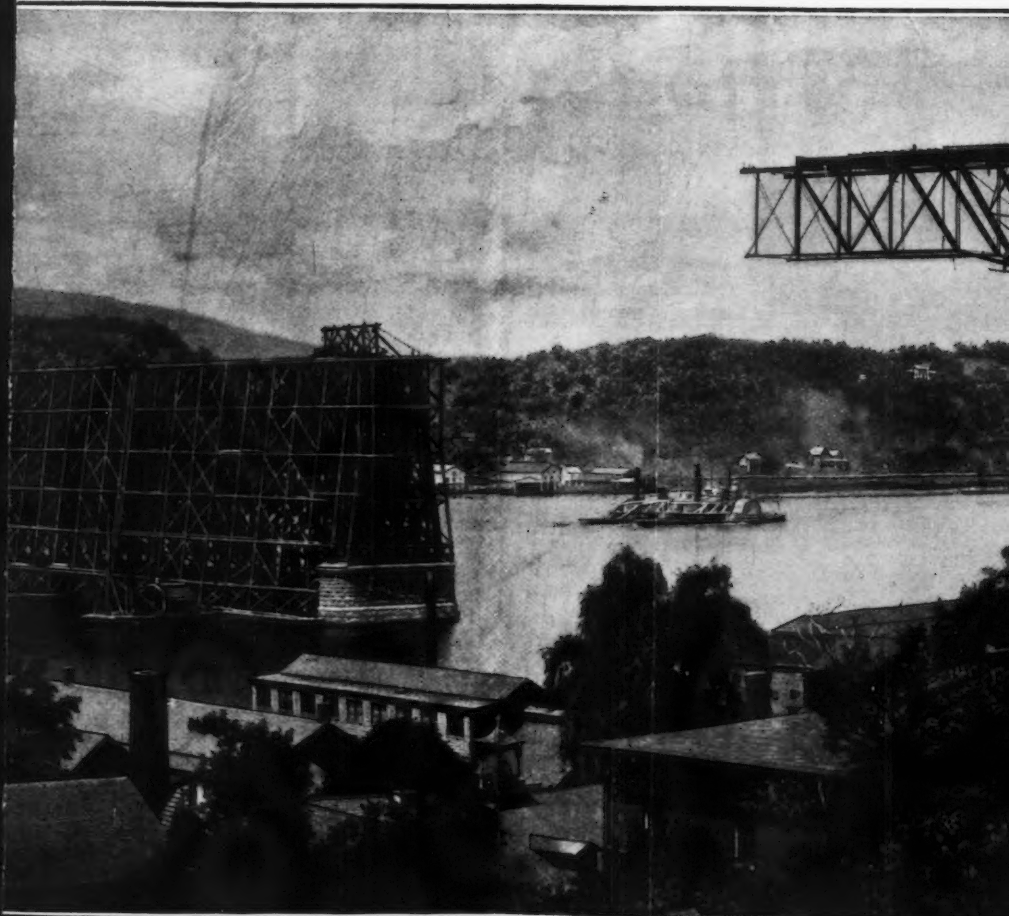
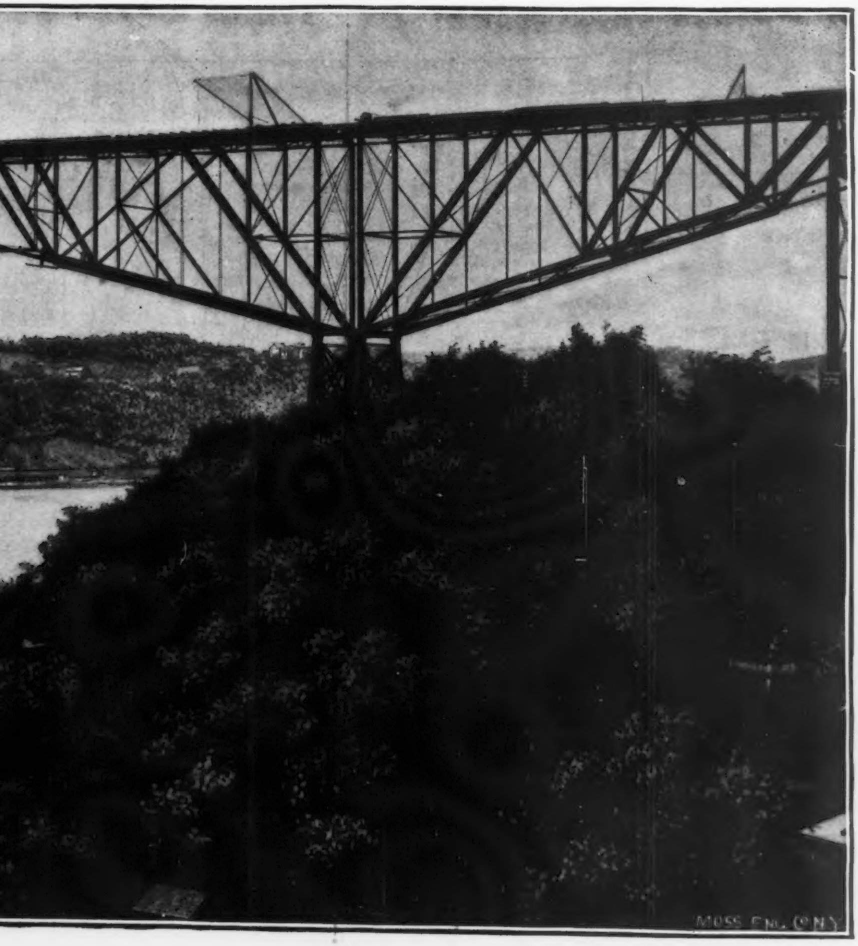


PLATE XLV.


MOSS ENG. CO N.Y.

as several could be tightened at one time, more equally, and to a greater strain than with men. This crib was towed out and made fast on the afternoon of May 25th. On that of the 26th it was placed exactly in position. No trouble was encountered at any time. The tide was not so bad as when Crib 4 was set, but the ebb was yet abnormally strong. The anchors were those used on Crib 4. They were lifted with a pile-driver, towed to their new positions and deposited without mishap. Notwithstanding the rough usage they had undergone, only one of those raised was found to be broken.

The dredging proceeded very evenly on Cribs 4 and 5. The daily progress of each, including building, loading and dredging, was about one foot per day. It is possible, by building during the day, and dredging and weighting by night, to double the speed.

This method of sinking foundations is applicable wherever mud, clay, or sand, has to be penetrated. While the bottom remains unsafe for founding on, it can be dredged, so there is practically no limit to the depth that may be attained.

It is not intended, in this paper, to treat of the design or manufacture of the superstructure, but some notes regarding the erection may be interesting. There are four sets of false-work for the main bridge, two sets for the shore arms, and two for the connecting spans. The former extend to the bottom of the upper chord. Their noteworthy features are great height, and the permanent character of the framing and bolting necessary to secure stability.

For the connecting spans, the depth of the water and the character of the river bed necessitate the use of compound piles 130 feet long. There are 22 bents of 24 piles each. They are made of two yellow-pine piles, with the butts dressed for a distance of 10 feet to an octagonal cross-section of 12 inches' diameter. These are spliced with eight pieces of spruce 4 x 5 inches x 20 feet long, fastened flatwise with $\frac{7}{16}$-inch boat spikes, 8 inches long, driven 1 foot apart. It occasionally becomes necessary to draw some of these piles. The bond is broken by additional driving. The first few blows upon the pile, with a 6 500-pound hammer and a fall of 4 feet, hardly move it, although when driven it went a foot with the final blow. When started in this way, it would hold together to be drawn out; and could then be suspended horizontally from the middle without showing less stiffness at the splice than elsewhere.

There are 528 piles for a span, arranged in sets of three under each post, firmly braced above low water, and fastened to the masonry at each end. The load on each pile will be about 5 tons, which the hammer test mentioned shows to be much less than its carrying capacity.

The caps of piling are 12 feet above high-water, and from them the trestle work extends to the bottom chord, an elevation of 130 feet. Details of this trestle are shown by accompanying drawings.

The deck of the false-work is occupied by four tracks. The two outer tracks are of 8-foot gauge, and upon these runs the large traveler for the erection of the span, which extends entirely across the space between these two outer tracks. The two rails next inside of these outer tracks are occupied by a hydraulic riveting apparatus, which spans the space, 19 feet wide, between these two rails. A single track of 4 feet, 8½-inch gauge at the center of the false-work, carries the cars conveying the material.

The traveler is composed of four bents, 95 feet high, 56 feet wide at the bottom, 62 feet at the top, and set 22 feet apart between centers. It encloses an opening 38 x 82 feet 6 inches, inside which the trusses are erected.

Some of the pieces to be handled weigh more than 20 tons each, so the derricks and rigging of the false-work and travelers must be proportionately strong. Two falls, of 16-inch triple-blocks, and 1½-inch rope, are used in raising the very heavy pieces. Some of the engines have six spools, and handle independently that number of lines.

The hydraulic riveter is carried on a small traveler, so spanning the two middle tracks as not to interfere with the transportation of materials on them. It consists of a boiler, engine and pump, by which power is supplied to the accumulator in the form of water supporting a heavy weight. The jaws are hung with differential pulleys from a worm gear set upon projecting timbers of the deck, which revolves like a turn-table. By this means the jaws are readily adjusted to any rivet in the bottom chord.

The towers at the ends of the span are first erected. Next, commencing at the fixed end, the bottom chord is laid along in place on camber blocks. The traveler then erects the span, commencing at the middle, and finishing each half successively.

From the shore arms and connecting spans 160-foot cantilevers are erected by means of projecting travelers, almost identical with those

Crib, Grillage and River Pier, Poughkeepsie Bridge. —

PLATE XLVI
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O'ROURKE ON
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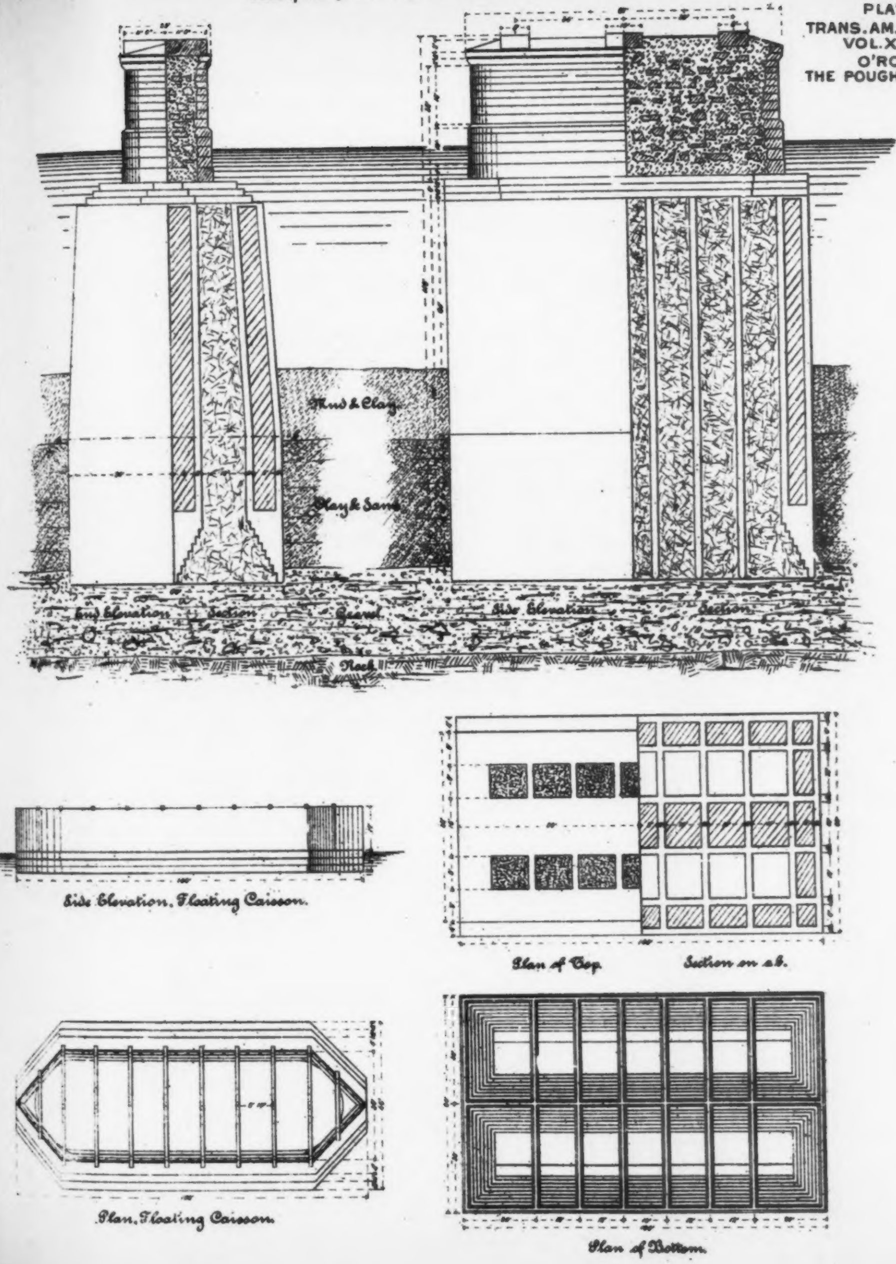

used for erecting the cantilever bridge across the Niagara River. They are composed of two trusses, 118 feet long, of which the chords and vertical posts are of wood, and the ties and splice-plates of iron. These are supported on a very heavy floor, extending from the rear end to within 50 feet of the other end, and are carried on twelve wheels, arranged in groups of one, two and three, respectively. Jack-screws, bearing on the front floor beam, relieve the wheels during the erection of a panel, and heavy hooks, under strong tension, clamp each end to the floor beams.

The 212-foot spans suspended from the ends of the cantilever arms are designed to be erected from the ends of the latter, and connected when they meet at the center. Stiff bottom chord, except in the middle three panels, enables each panel, when finished, to support the traveler during the erection of the next beyond. When the travelers meet, the remaining three panels are completed. In connecting the center panel, the top opening should be a little long in order to let the chord section into place, and the bottom one a little short, that the pins may drive easily into the eye-bars. This is insured by adjustment struts between the arms and suspended span, which are shortened in the top chords, and lengthened in the bottom by rollers separated with wedges. They are so arranged, that by drawing them the ends of the top chord approach each other, and those of the bottom chord recede. After the span is connected, the wedges and rollers are removed, and the trusses hang suspended.

The illustrations accompanying this paper are as follows:

PLATE XLVII
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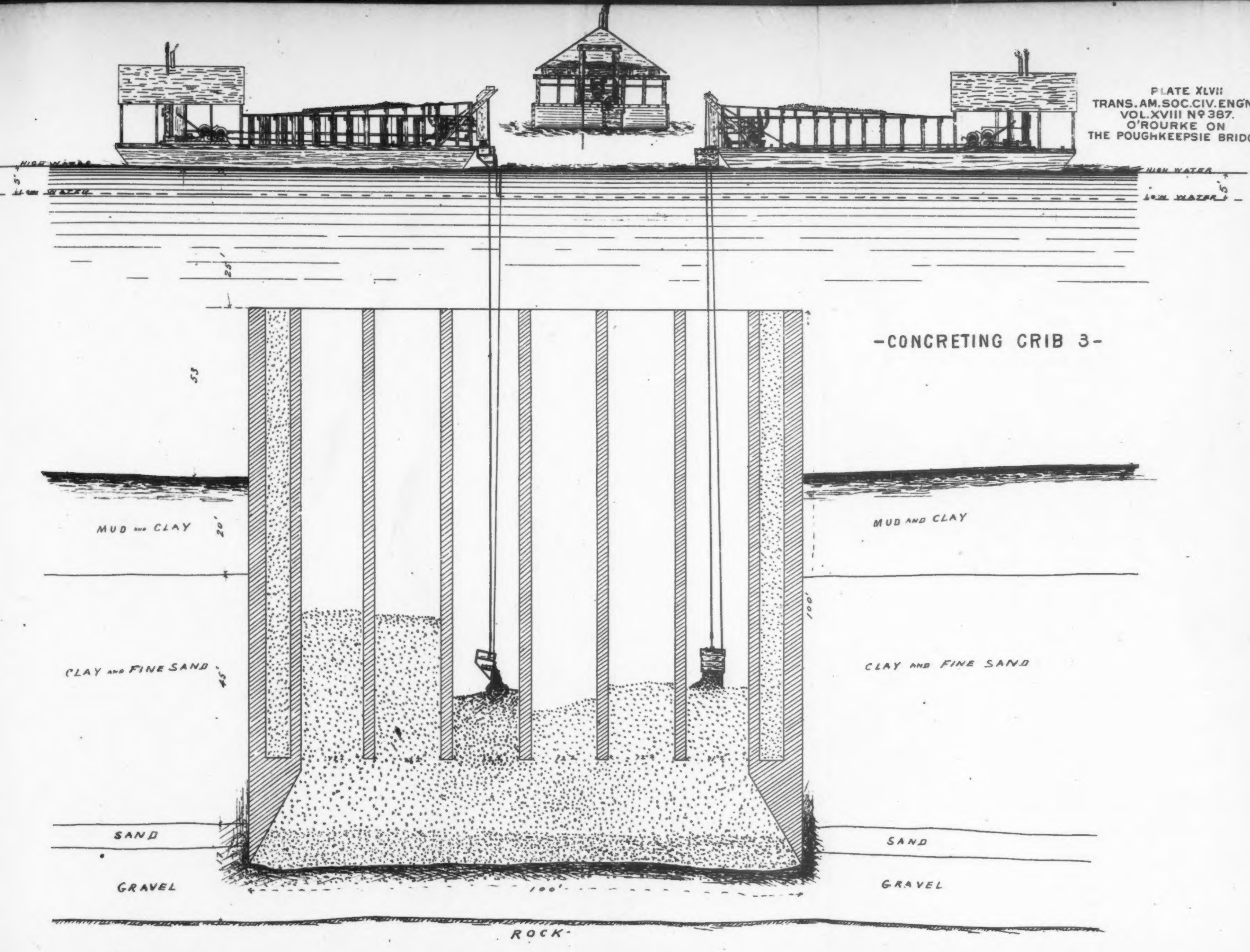
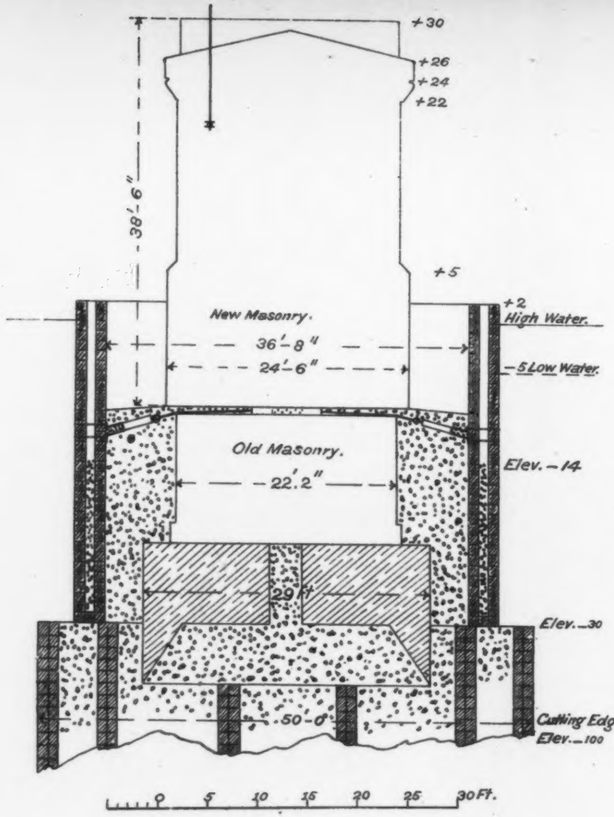
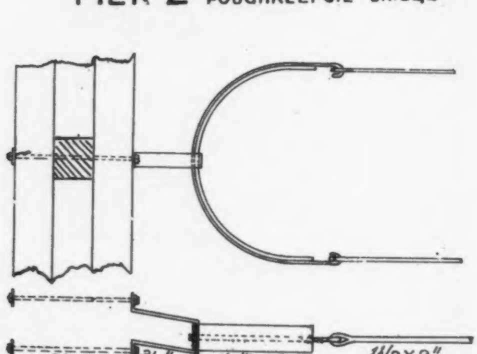
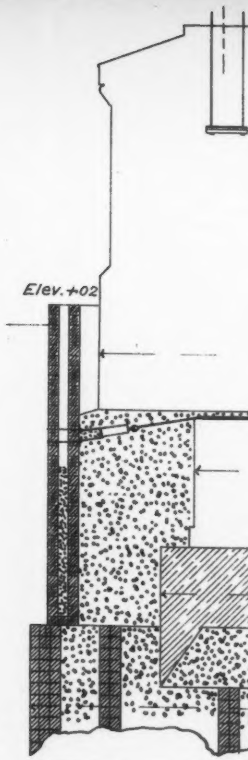
THE POUGHKEEPSIE BRIDGE.





PIER 2 POUGHKEEPSIE BRIDGE





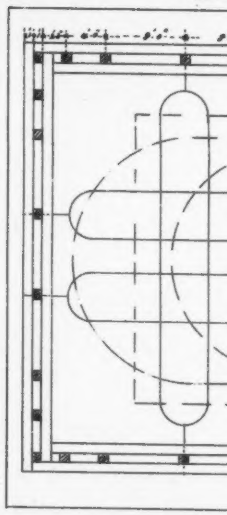


PLATE XLVIII
TRANS. AM. SOC. CIV. ENGNS.
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O'ROURKE ON
THE POUGHKEEPSIE BRIDGE.

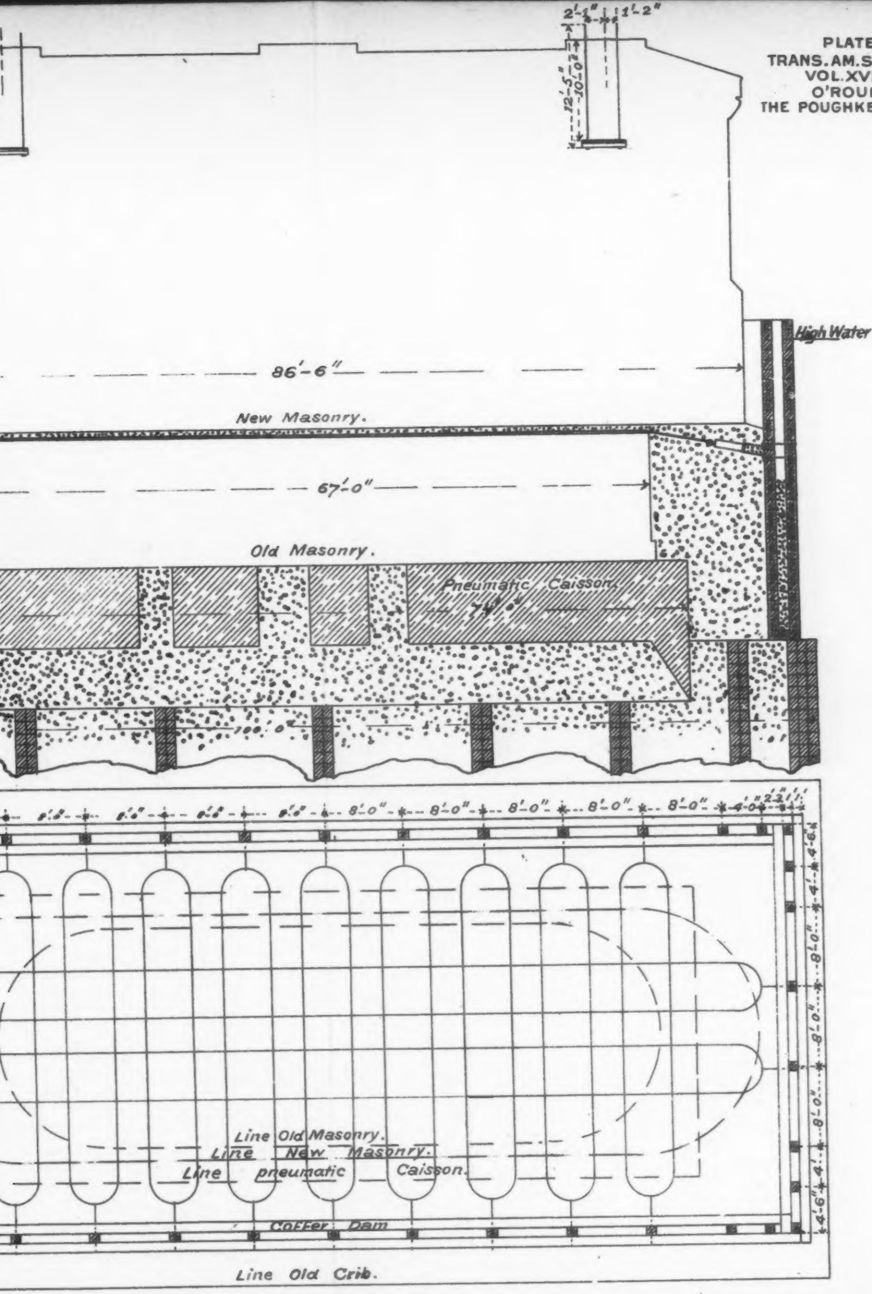

PLATE XLIX
TRANS. AM. SOC. CIV. ENG'NS.
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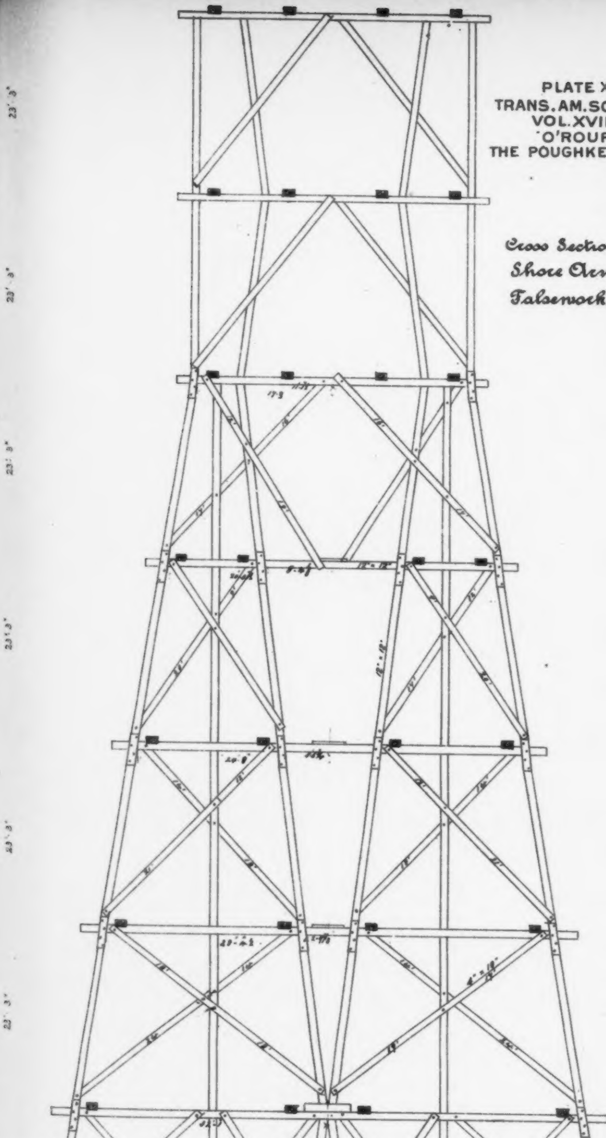
O'ROURKE ON
THE POUGHKEEPSIE BRIDGE.



Cross Section
Shore Arm
Falsework.

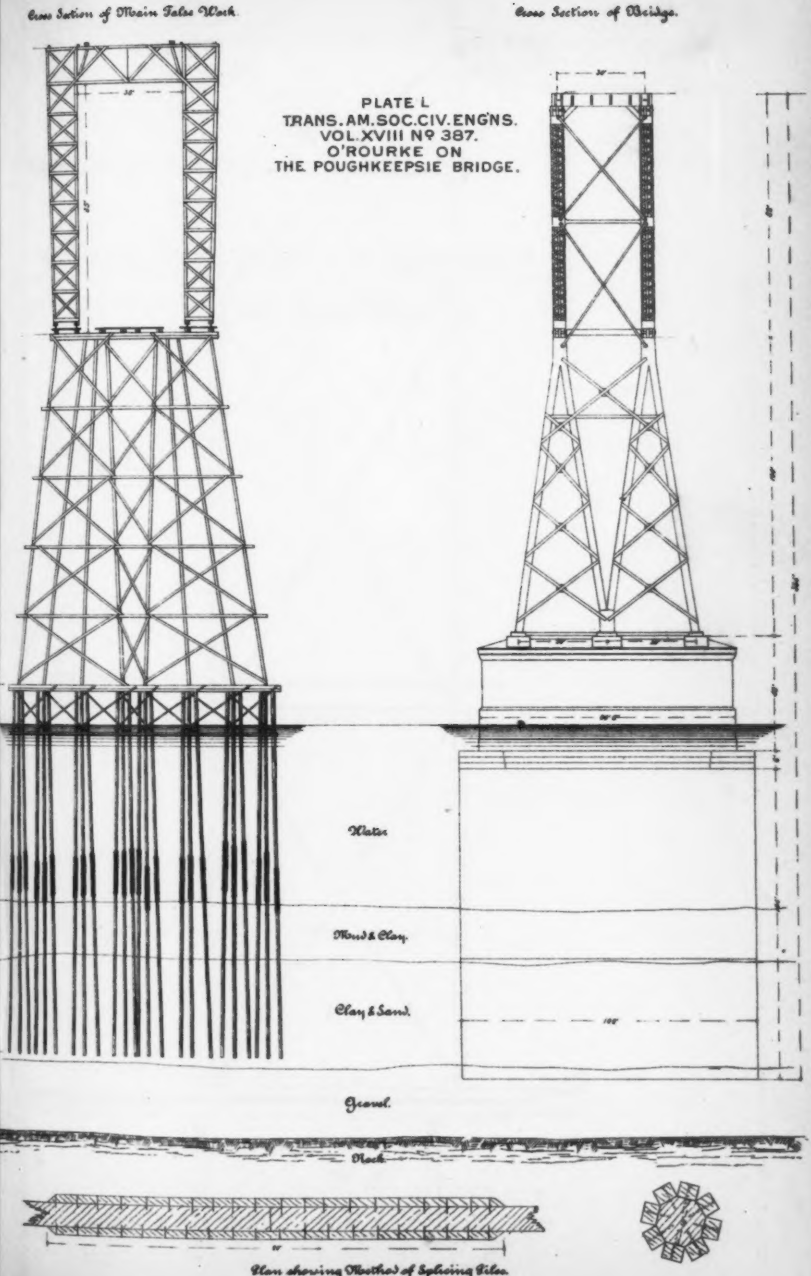


PLATE L
TRANS. AM. SOC. CIV. ENG'NS.
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O'ROURKE ON
THE POUGHKEEPSIE BRIDGE.

PLATE L
TRANS. AM. SOC. CIV. ENG'NS.
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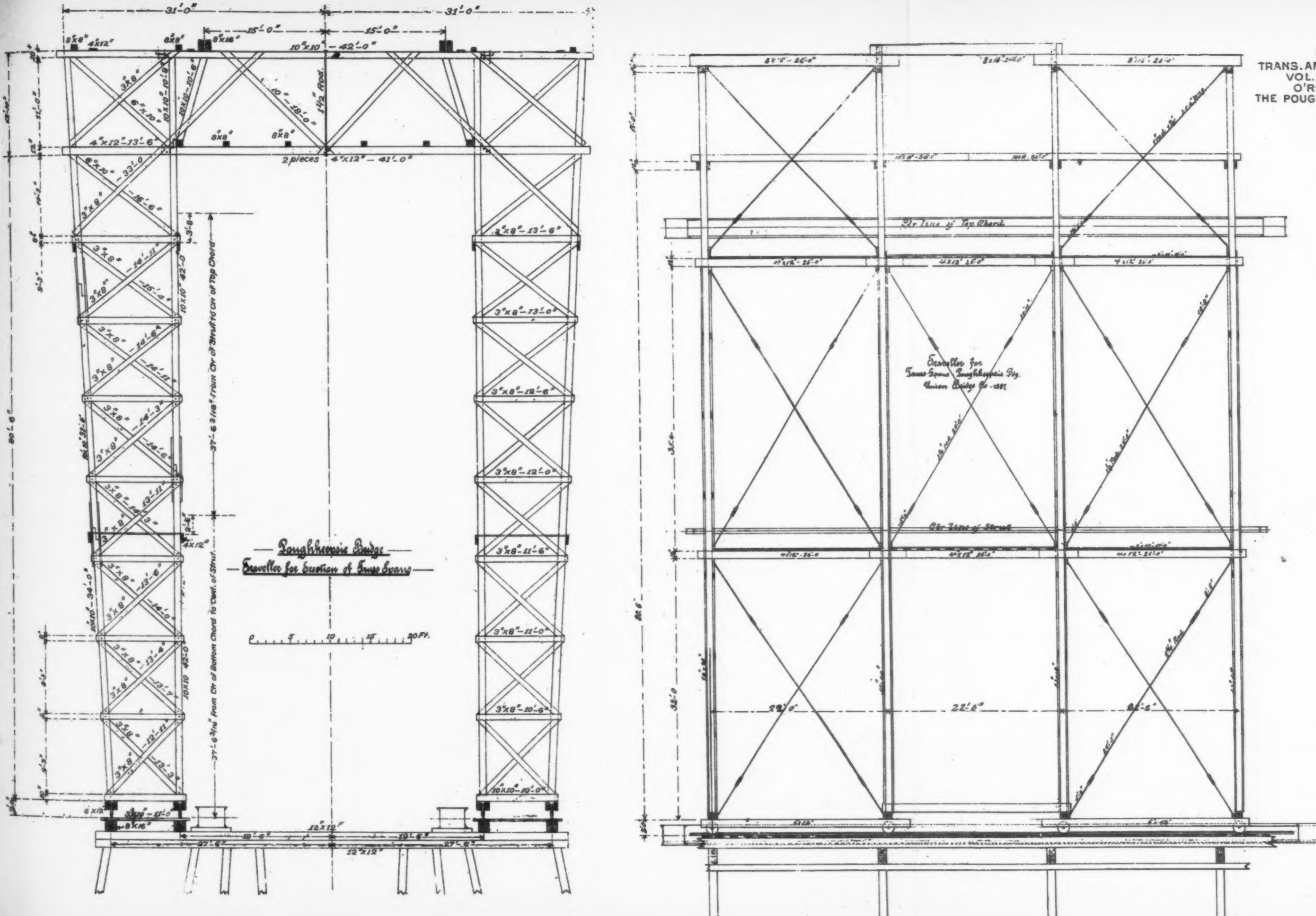
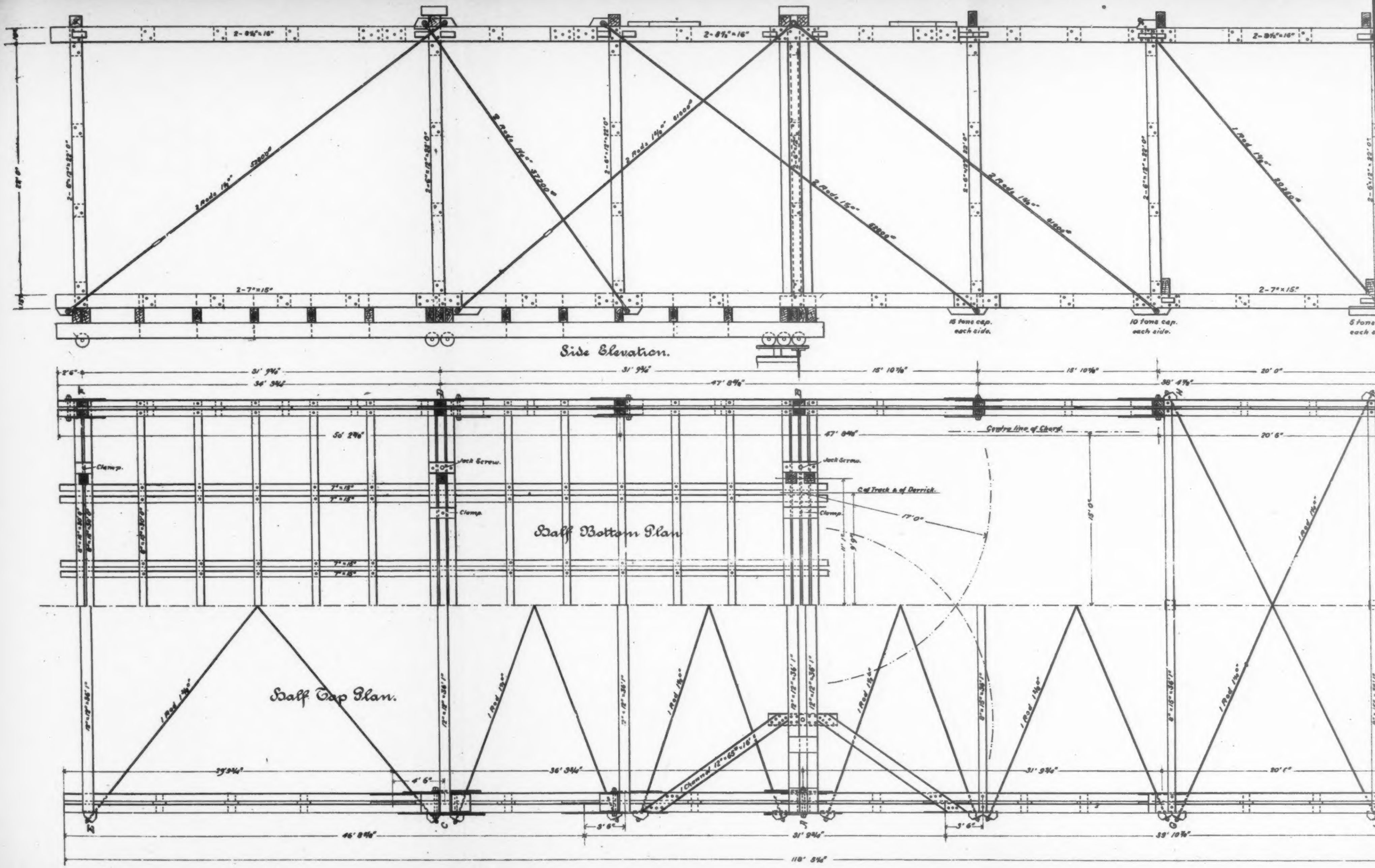
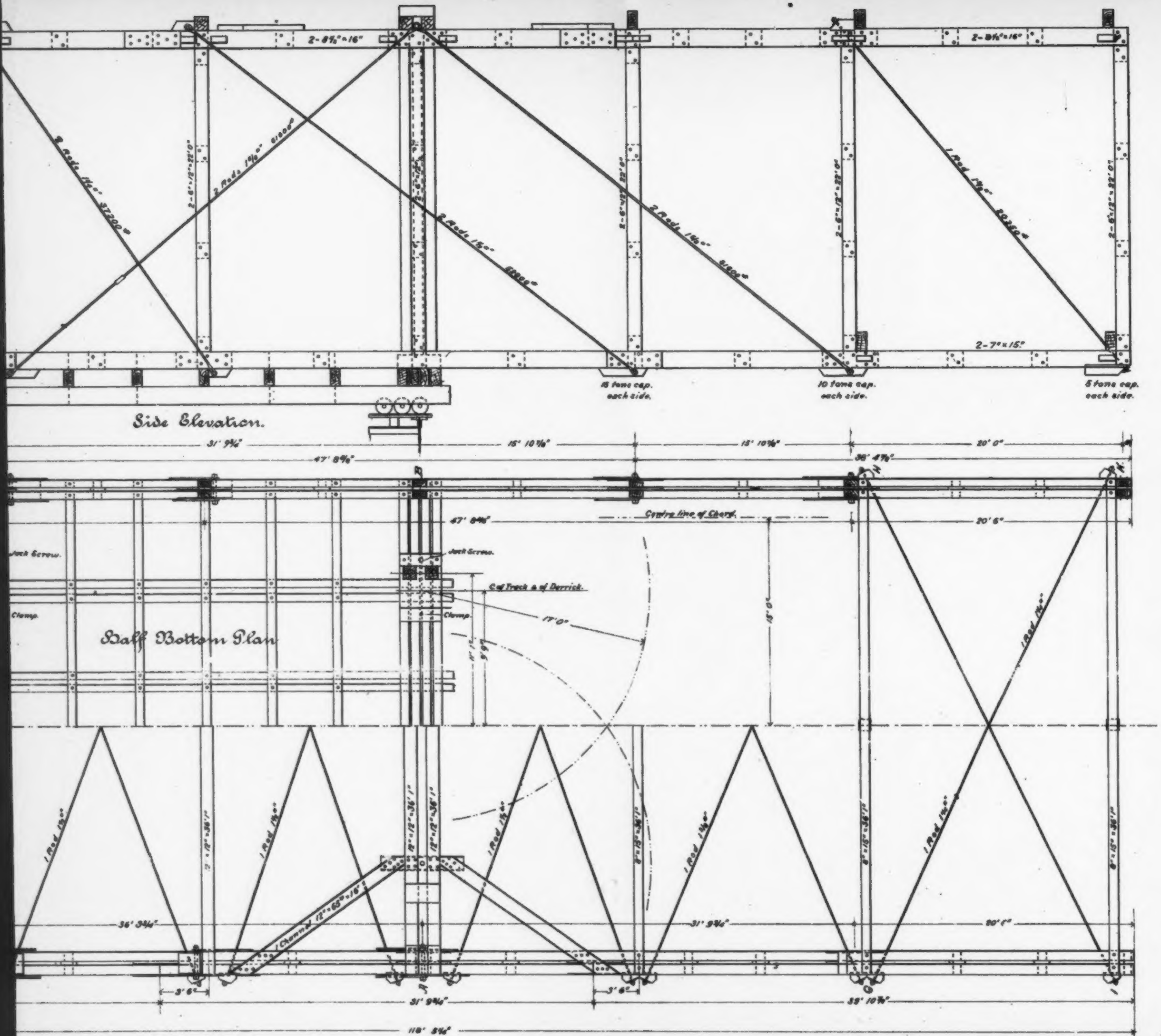
THE POUGHKEEPSIE BRIDGE.





Side Elevation.
Half Bottom Plan
Half Top Plan.



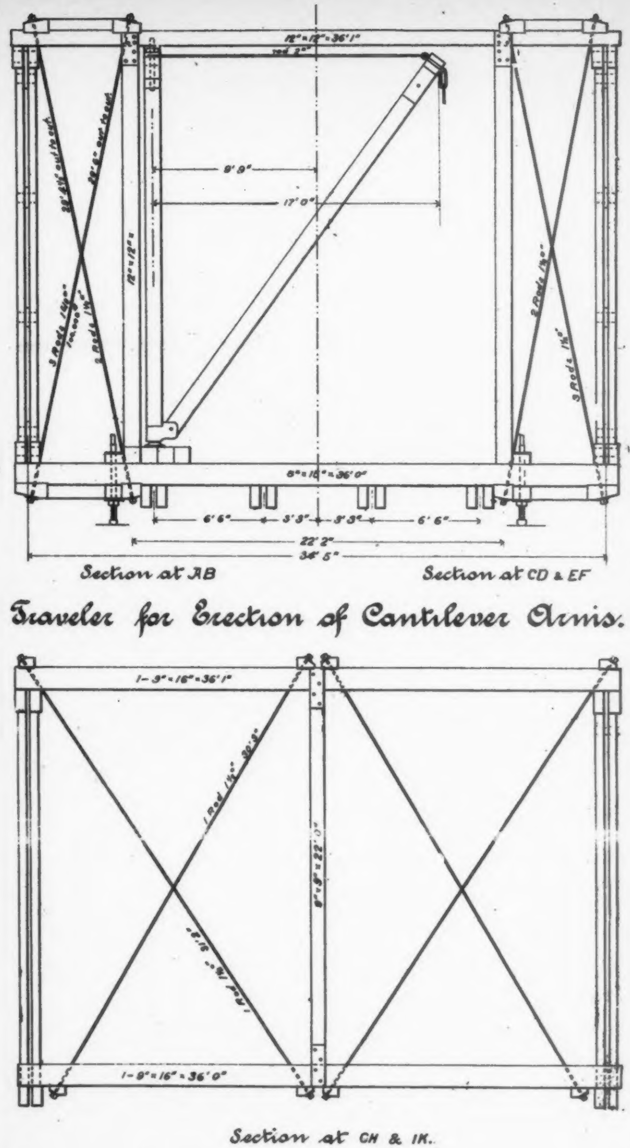


PLATE LII
TRANS. AM. SOC. CIV. ENGRS.
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O'ROURKE ON
THE POUGHKEEPSIE BRIDGE.

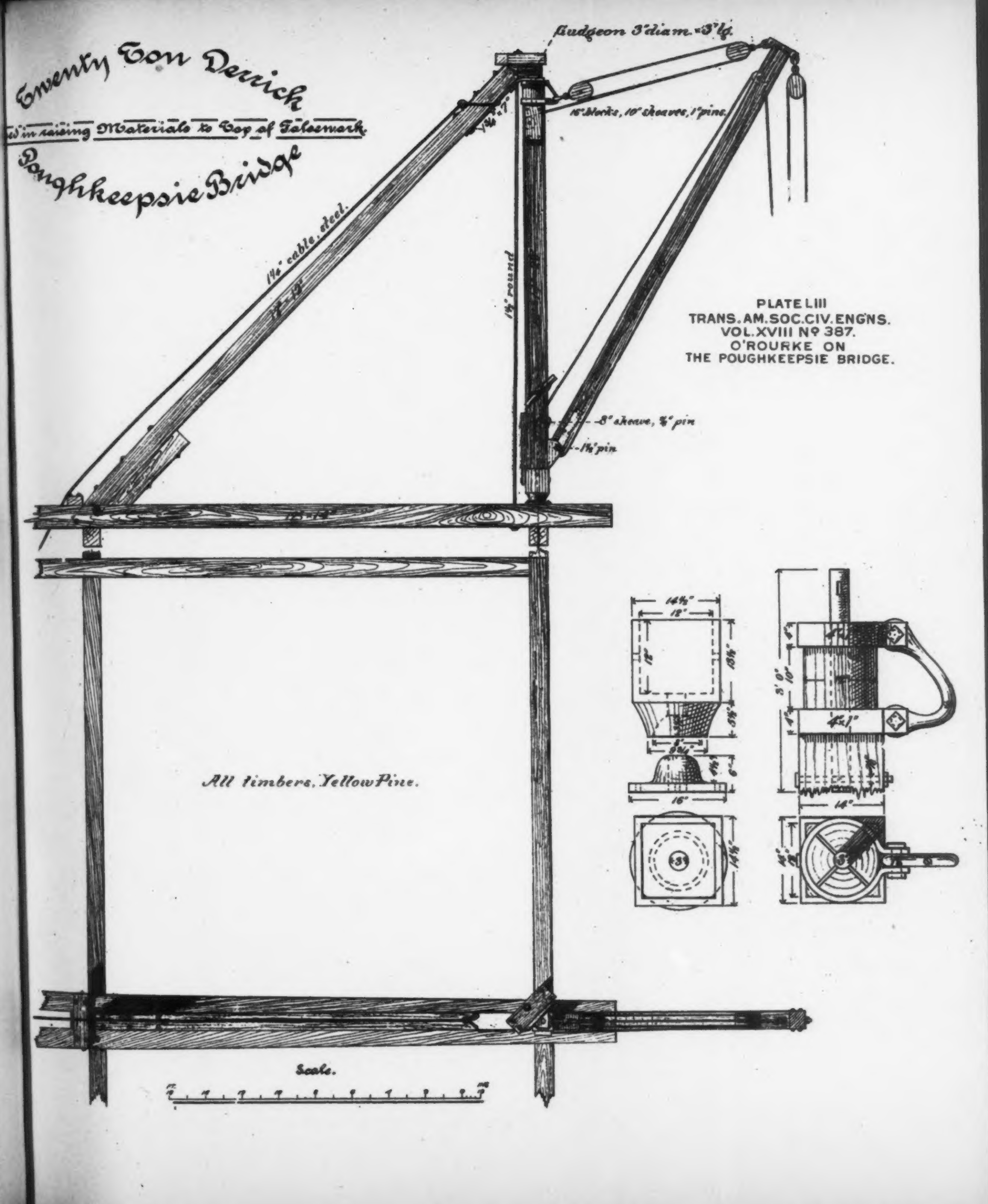

PLATE LIII
TRANS. AM. SOC. CIV. ENGNS.
VOL. XVIII Nº 387.
O'ROURKE ON
THE POUGHKEEPSIE BRIDGE.

PLATE LIV
TRANS. AM. SOC. CIV. ENG'NS.
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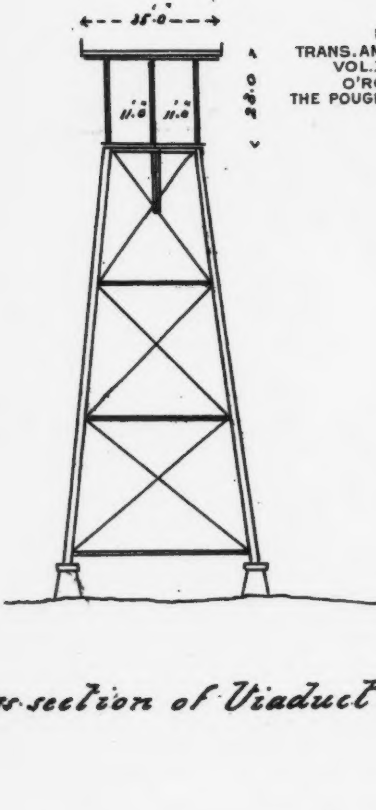
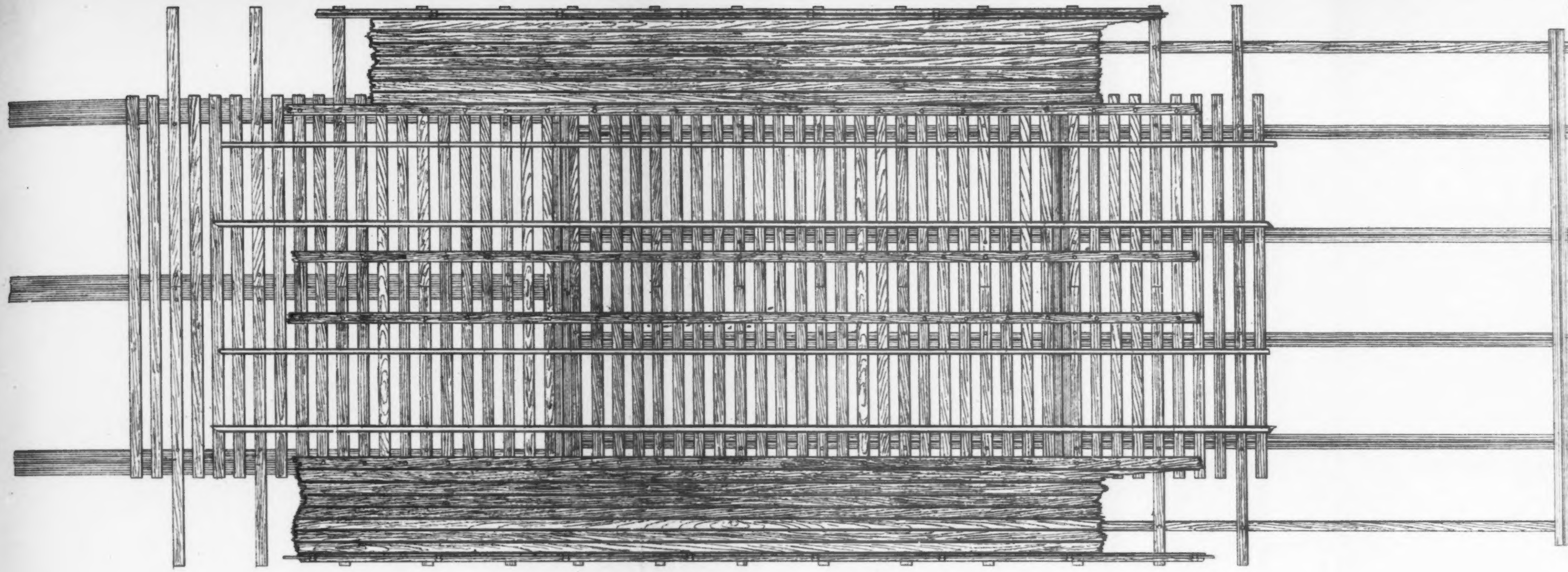
O'ROURKE ON
THE POUGHKEEPSIE BRIDGE.



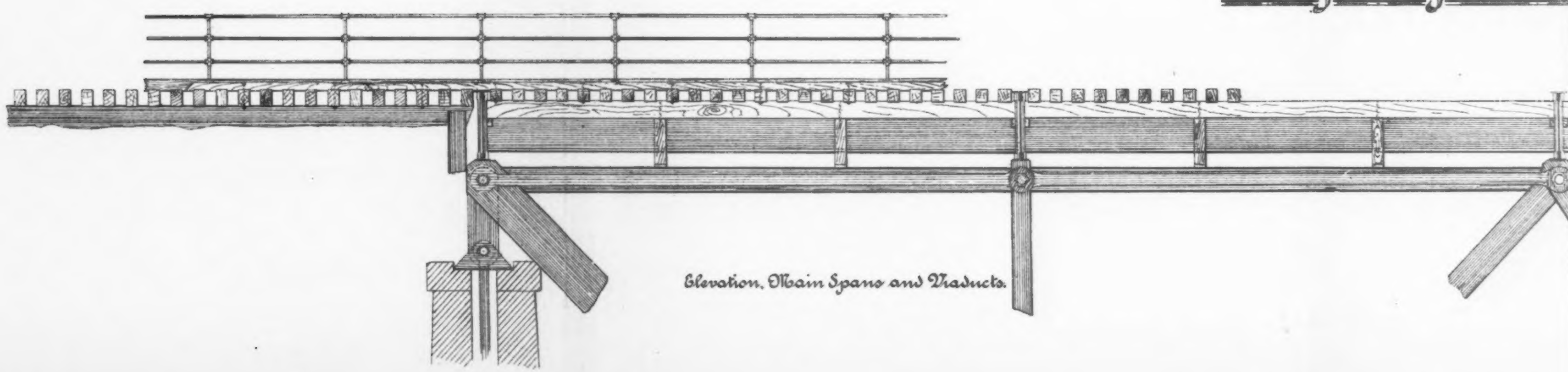
Cross-section of Viaduct



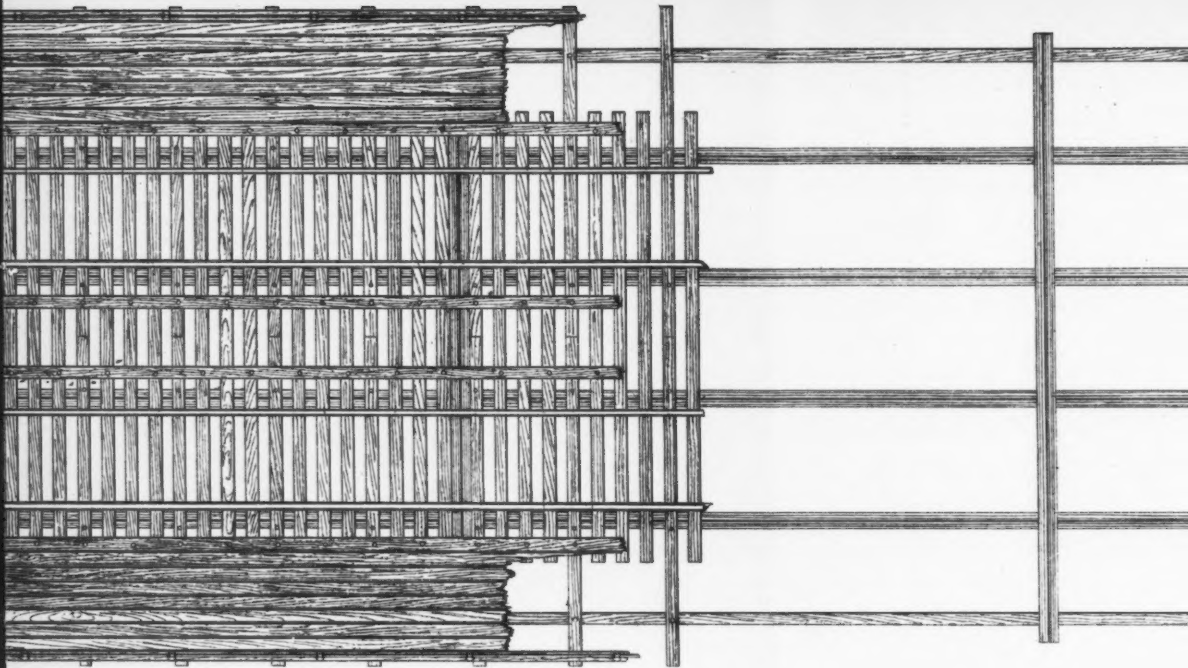
Plan, Main Spans and Viaducts.

General Plan of Floor S

Poughkeepsie B

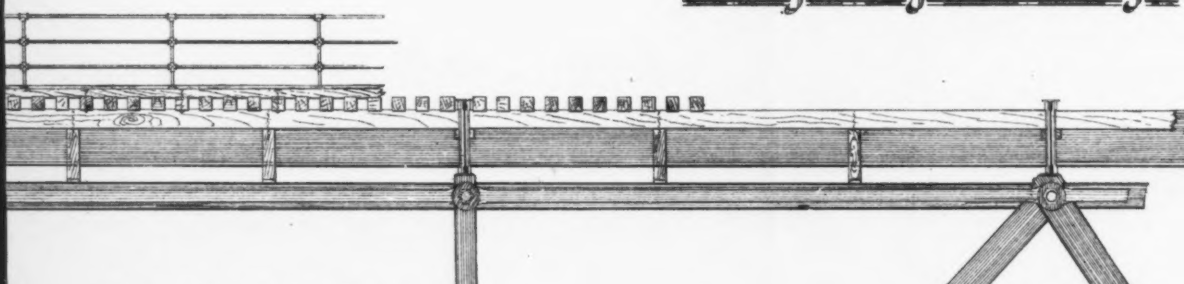


Elevation, Main Spans and Viaducts.



Plan, Main Spans and Viaducts.

General Plan of Floor System
Poughkeepsie Bridge.



Elevation, Main Spans and Viaducts.

PLATE LV
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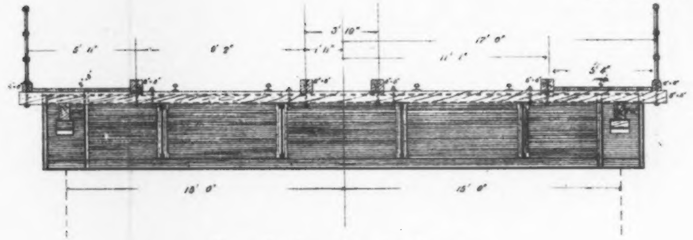
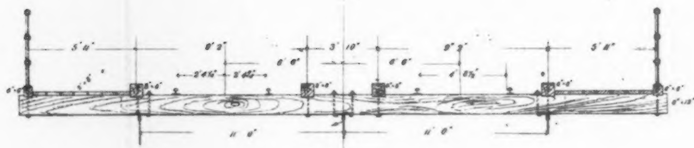
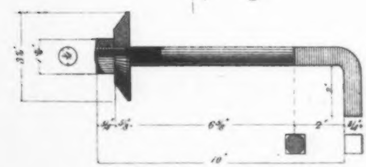
O'ROURKE ON
THE POUGHKEEPSIE BRIDGE.



Cross Section, Main Spans.



Cross Section, Viaducts.



Hook Bolt, Scale 4" = 1'.

Iron — Bill of Material. — Timber.

Location.	No.	Description.	Size	Weight	Remarks.	Description.	Pieces	Length	Section	Ft. B.M.	Remarks
Main Spans.	4750	Guard Rail Bolts.	3/4" × 16" under head @ 2.55	12100	1 cast & 1 stamp washer. Head under.	Stringer Posts.	392	3' 0"	8" × 8"	6272	
Viaducts.	2800	- - -	3/4" × 17" - - - 2.68	7500	2 stamp washers - -	Stringers.	148	32' 0"	8" × 12"	37888	size to 11½"
-	2800	- - -	3/4" × 19½" - - - 3.00	8400	1 cast, 1 stamp washer - -	-	42	30' 0"	-	10080	- - -
Main Spans.	1180	Stringer -	3/4" × 21" - - - 3.18	3750	1 cast washer. Head driven flush with tie.	-	4	20' 0"	-	640	- - -
- -	1180	Hand Rail -	3/4" × 14" - - - 2.31	2700	1 cast, 1 stamp washer. Head under.	-	8	16' 0"	-	1024	- - -
Viaducts.	1380	- - -	3/4" × 18" - - - 2.70	3750	do.	Ties.	2070	24' 0"	-	397440	- - suit girder
Main Spans	2400	Hook -	10" - 2.00	4800	3/4", 8" (from end) cast 3/4", 1 cast washer.	-	1380	17' 6"	-	193200	- - - -
Viaducts.	2800	- -	14" - 2.25	6300	3/4", 12" - - - - -	-	1768	24' 0"	8" × 8"	226304	- - 7½"
Main Spans.	400	Drift -	3/4" × 20" - 2.50	1000	For Stringer Posts.	-	1180	17' 6"	-	110133	- - -
	16000	Stamp Washers	3/4" bolts. - -	16000		Guard Rail 28500 lin. ft.		16' to 24'	-	152000	- - -
	17000	Cast -	- - - 0.33	5700		Footing Rail.	640	22' 0"	6" × 6"	42240	- - 5½"
		30 d. Nails.		3000		Plank	6000	16' 0"	2" × 8"	128000	
Weight of Head & Nut			0.55	73000		Total for Bridge				1305221	Yellow Pine.

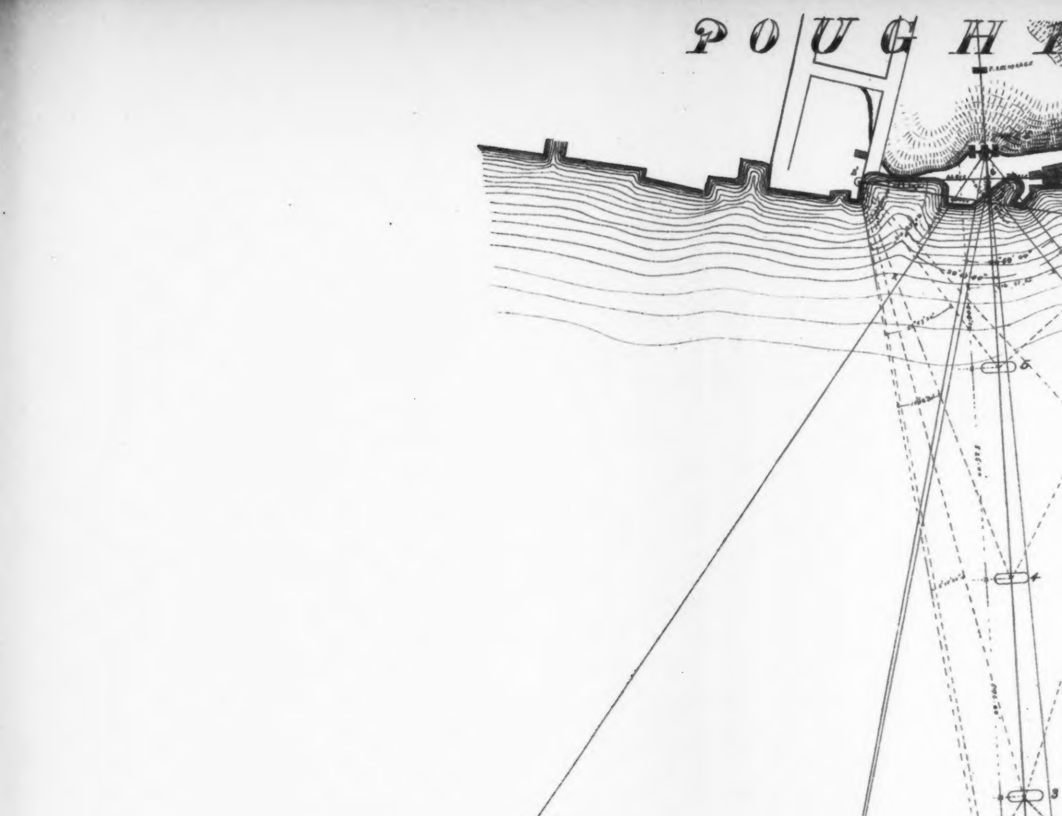

POUGH
ULSTER

PLATE LVI
TRANS. AM. SOC. CIV. ENG'NS.
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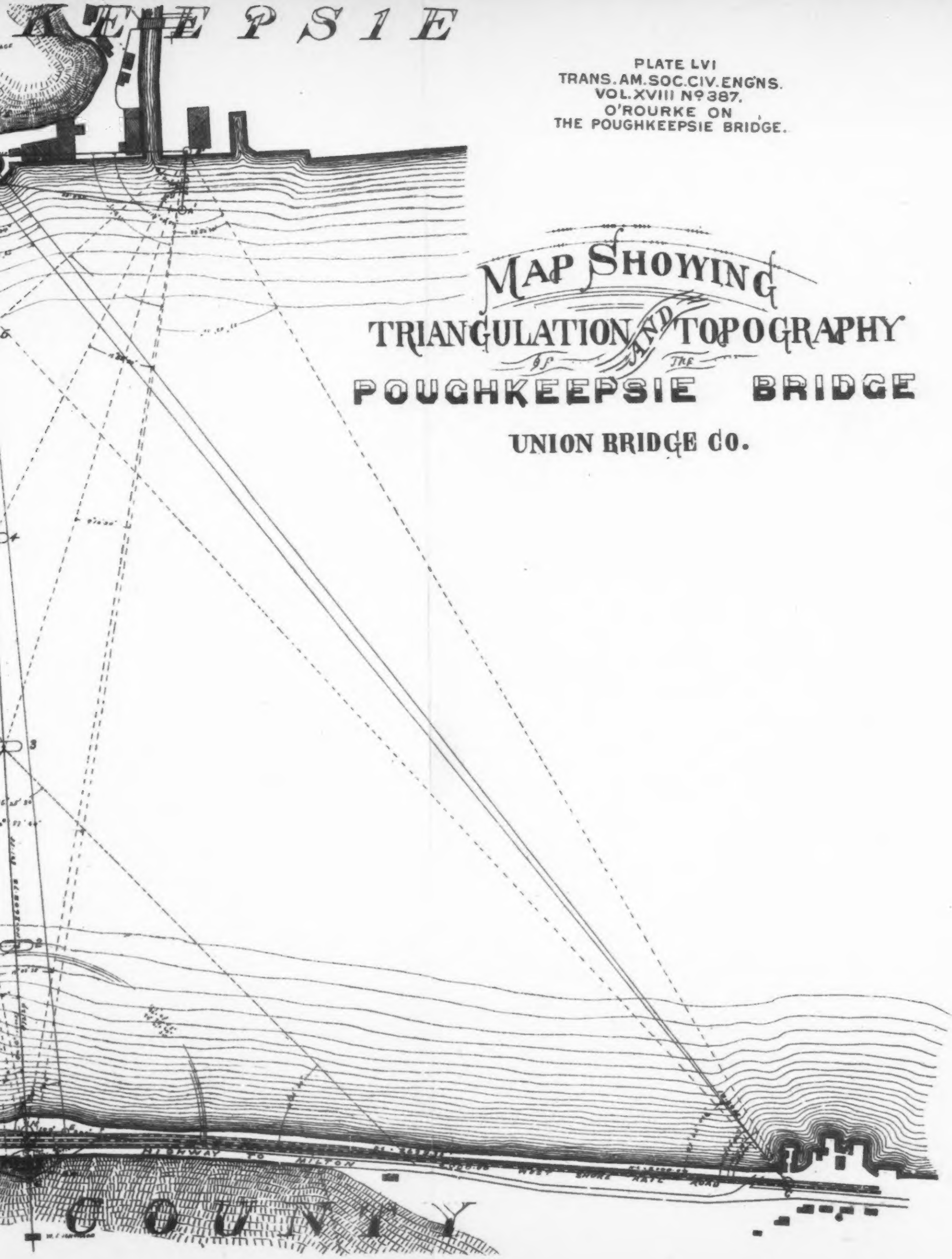
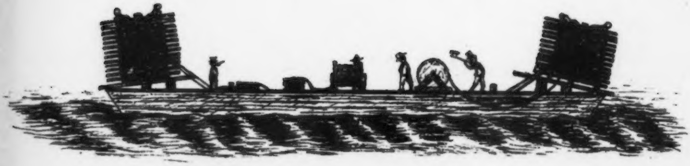
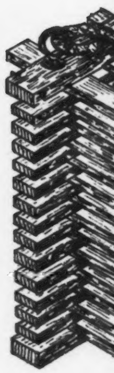
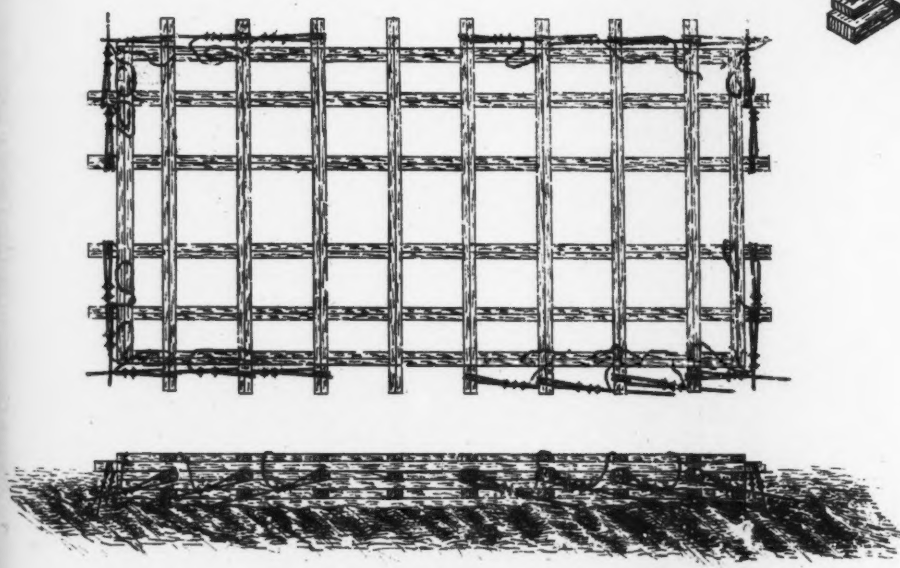
O'ROURKE ON
THE POUGHKEEPSIE BRIDGE.





METHOD OF LAUNCHING ANCHORS





CRIB, ANCHORED IN POSITION

PLATE LVII
TRANS. AM. SOC. CIV. ENG'NS.
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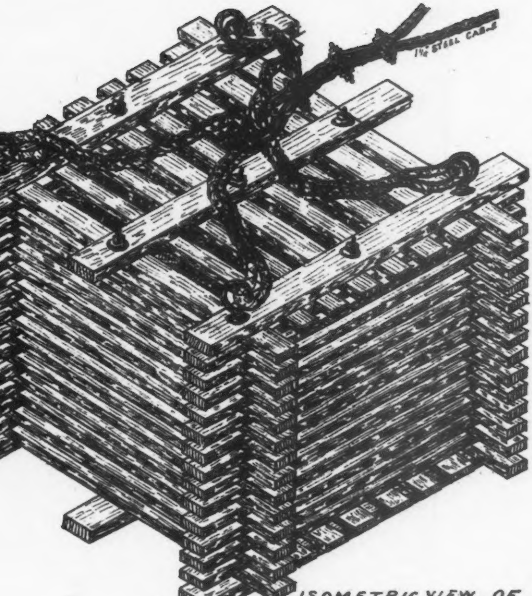
O'ROURKE ON
THE POUGHKEEPSIE BRIDGE.

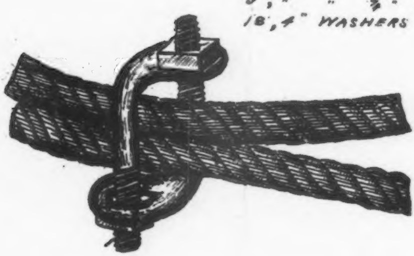


ISOMETRIC VIEW OF CHINEESE ANCHOR

6' x 6' x 7' INSIDE MEASUREMENT LOADED WITH BROKEN STONE

BILL OF MATERIAL

52 HEMLOCK STRIPS 3" x 6" x 11'
6 " " 4 x 9" x 11'
12 " " 3 x 4 x 10'
4, IRON RODS 7/8" DIA. 7'5" BTW. WASHERS - EYE AT ONE END
5, " " 3/4" " 7'5" " "
18, 4" WASHERS



STYLE OF CLIPS USED

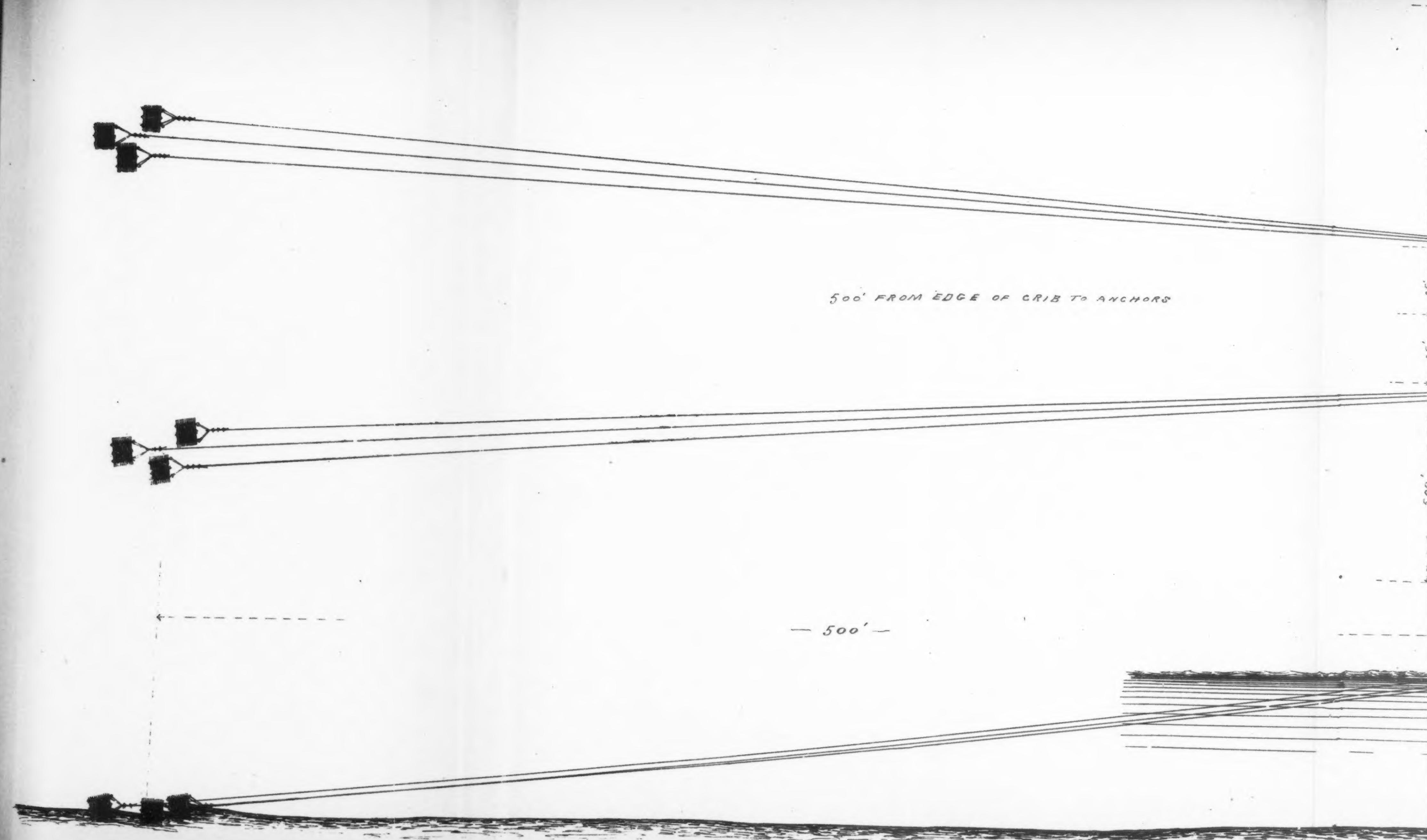

500' FROM EDGE OF CRIB TO ANCHORS
— 500' —
25'
25'
500'

PLATE LVIII
TRANS. AM. SOC. CIV. ENG
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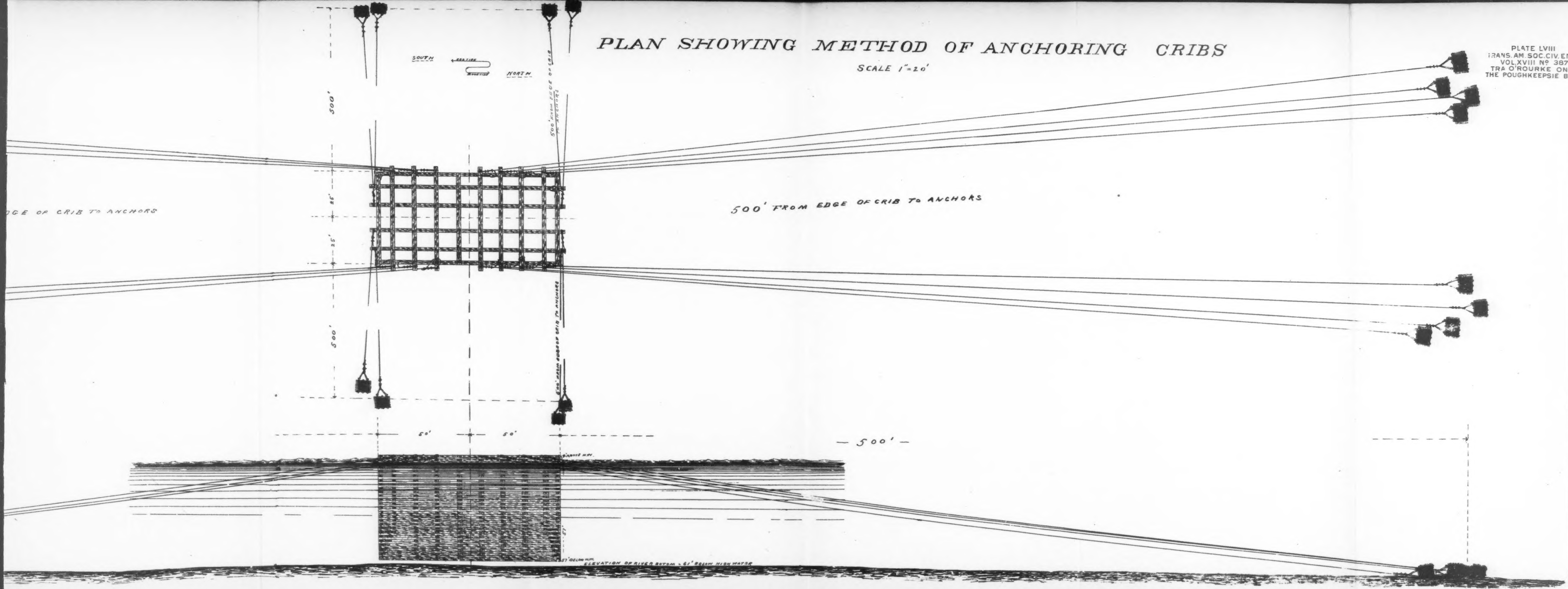
TRA O'ROURKE ON
THE POUGHKEEPSIE BR

PLAN SHOWING METHOD OF ANCHORING CRIBS

SCALE 1" = 20'



PLAN SHOWING METHOD OF ANCHORING CRIBS

SCALE 1"=20'

PLATE LVIII
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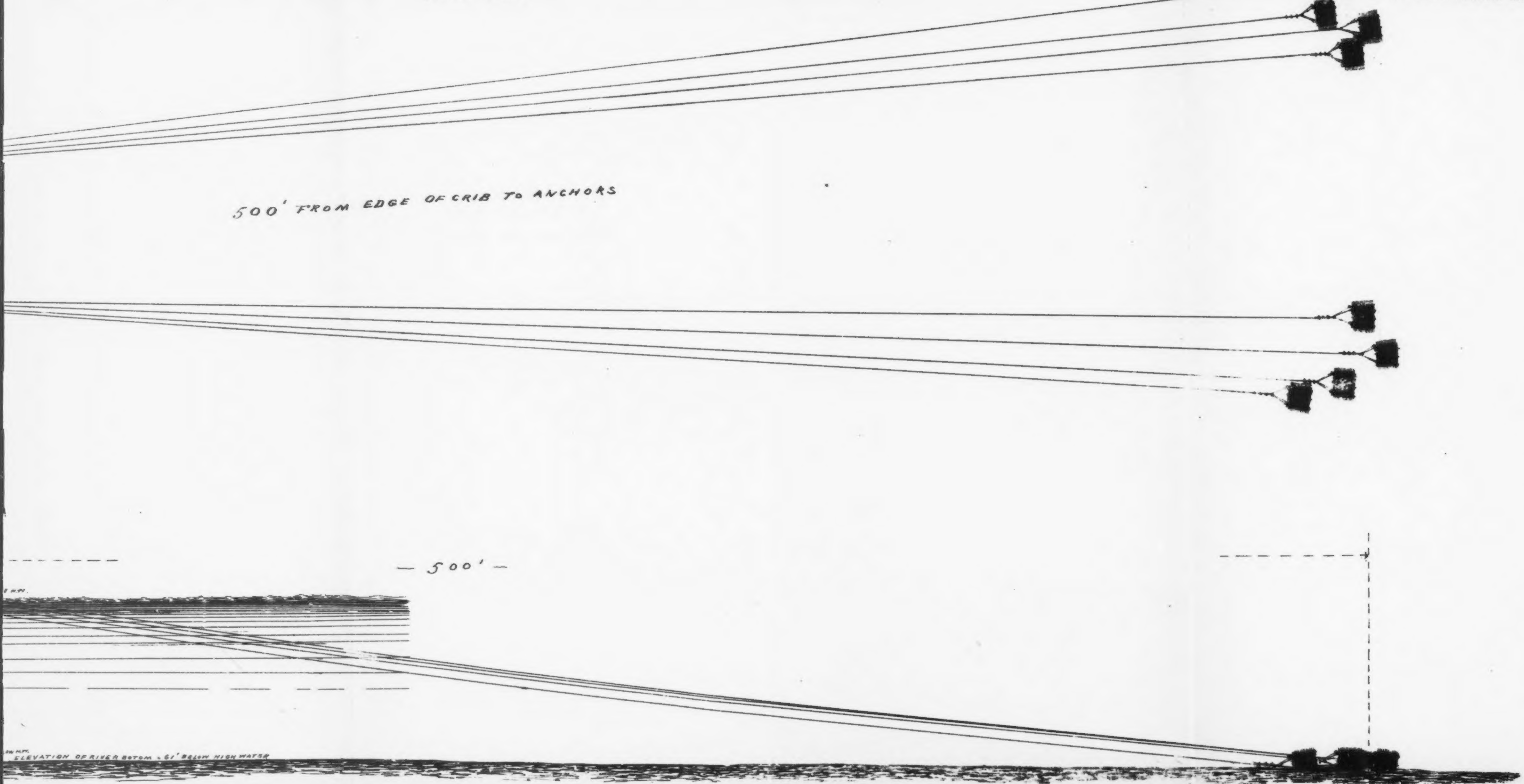
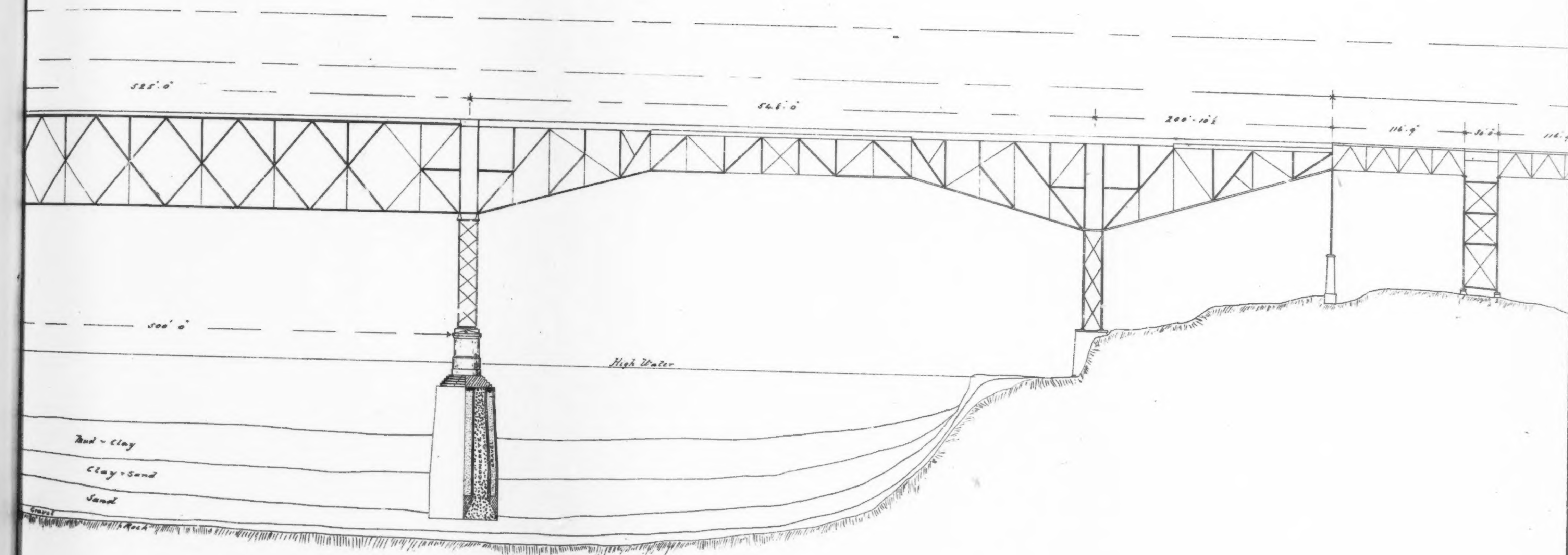
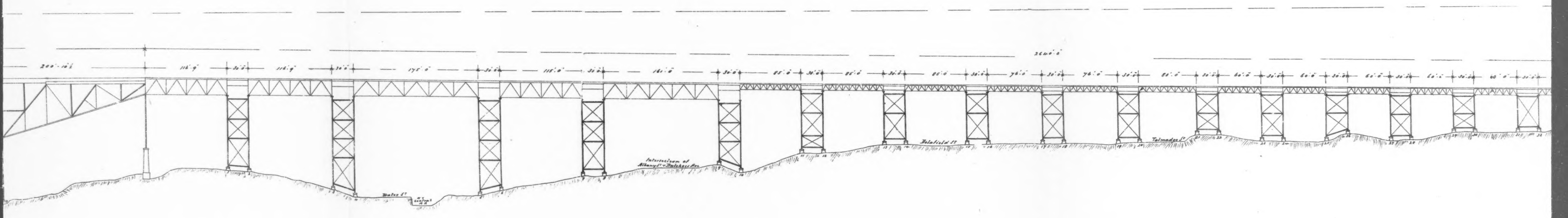
TRA O'ROURKE ON
THE POUGHKEEPSIE BRIDGE.




525' 0"
548' 0"
200'-10½"
116' 9"
30' 0"
500' 0"
High Water
Mud & Clay
Clay & Sand
Sand
Gravel
Rock
CONCRETE
GRAVEL

GENERAL PLAN OF POUGHKEEPSIE BRIDGE



EEPSIE BRIDGE

PLATE LIX
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THE POUGHKEEPSIE BRIDGE.